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JOURNAL
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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JOURNAL
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EXPERIENCES WITH WATER WHEEL UNIT ALIGNMENT

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INTRODUCTION

Within recent years several papers have been presented which have carefully and thoroughly outlined satisfactory methods and procedures for the erection of water wheel generator units which should result in the original alignment of these units being very nearly true. Proper original alignment, however, does not necessarily assure continuing alignment or freedom from operating troubles which depend wholly or in part upon alignment. While there is evidence that present day power plants with modern methods of concrete control applying to their construction will probably "stay put", experience with earlier construction shows that power house foundations settle, substructure concrete grows, construction joints and cracks are filled with silt, electrical faults and equipment overloads result in the shifting on base plates so that it seems desirable to have a simple convenient method of periodically checking alignment to forestall troubles.

In this paper an attempt has been made to collect and correlate some of the typical proven cases of misalignment experienced by American operating companies after original erection, to study the causes of this behavior, to note the methods used in correcting them and to suggest a method for checking a misalignment condition. At first thought, it might seem that there have been, and probably will be in the future, so few cases of this type of trouble that no additional expense in the original power house layout is warranted to "facilitate a job which may never have to be done". This is brought out by a statement made by one operating company: "We have some 108 units in 56 power houses and feel that problems allied with misalignment have not been a factor in the operation and maintenance of our hydro electric system." And again from another large operator, "In the past 15 years we have realigned a total of six vertical reaction units. We have records which would indicate that these units were properly aligned at the time of installation and with the exception of two units the realignment was done as an incidental part of unit rebuild and was not required to correct conditions which made normal operation impossible."

Note: Discussion open until September 1, 1956. Paper 947 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 2, April, 1956.

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"With about 25 hydro plants on another system, some dating back about fifty years, there has never been a single case of misalignment of units due to concrete growth. In one small single unit station, however, there has been a slight tilt of the whole power house structure due to foundation settlement but no misalignment of bearings and no trouble has been experienced in operation."

Tennessee Valley Authority and Bureau of Reclamation also report respectively: "We have had no difficulty at all with any of our units following their initial proper alignment extending over a period of over twenty years with more than 75 hydro electric units ranging in size from 5000 to 67000 KW."; and, "We have experienced no trouble in this regard."

On the other hand there have been a considerable number of cases of progressive misalignment—quoting one company, "We have experienced shifting of alignment as early as two years after the development had first been put on the line."; and another, "It is very unfortunate that design and erection engineers do not generally make sufficient provision for those responsible for operating and maintaining large hydro generators to make periodic checks on alignments and levels without dismantling a large part of the unit, and consequently serious troubles generally develop before there is knowledge of a change of alignment."

Because of this experience we must give the possibility of misalignment consideration in our design and in our maintenance procedures. The principal cause of the troubles appears to be concrete growth which may not become apparent for many years. It is not within the scope of this paper to outline the chemistry of its operation but only to indicate that it has occurred, that it has occurred to an extent requiring major corrective measures, and to suggest a method of following its progression so that remedial measures may be taken at the proper time. Neither does this paper suggest the remedial measures to be taken nor the design practices now used to reduce or eliminate concrete growth in a new structure. There have been, and undoubtedly will be, other cases of misalignment caused by electrical disturbances, rock squeeze in deep cuts or even earthquakes.

History

The following brief descriptions of a few of the characteristic experiences and the corrective measures devised will be of value in pointing the way ahead.

At Conowingo Station (Maryland) with 7 units, after twelve years of operation, growth of the surrounding concrete encasing the 27 foot butterfly valves immediately ahead of the scroll case resulted in an out-of-round condition which required periodic grinding of the bronze seals. When no further grinding was possible, the valve casings were freed from the concrete surrounding them and further distortior eliminated. At about the same time it was discovered that the vertical clearances between the bottom runner band and the lower distributing ring did not follow the clearances at the top of the runner, and that a change in position was taking place at the bottom of the speed ring, all of which was diagnosed as a movement caused by concrete growth.

After 21 years of operation one unit was completely dismantled with considerable difficulty because of binding of parts and the following conditions found: the speed ring was distorted so that the lower cast iron distributing ring showed evidence of runner rubbing and was out-of-round and cracked requiring metal lock repair and machining both inside and outside to a maximum

of .25 inches. The head cover also required machining to reduce the diameter by .375 inches. The generator sole plates and the thrust bearing runner plates were found out of level which was believed partly due to the concrete growth.

At Safe Harbor Station (Pennsylvania) with 7 units no appreciable vertical expansion or growth of concrete has been observed but a horizontal growth has taken place over a period of from 8 to 17 years resulting in the distortion of the water wheel throat rings from a circular to an eliptical shape. This results in decreased clearances between the throat ring and the runner, which has a diameter of 18.3 feet, until rubbing actually occurs. When rubbing does occur, adjustments of the thrust and guide bearings will suffice for a time but eventually grinding of the throat ring has been necessary to as much as .025 inches.

Recently the guide bearing of a large overhung vertical single phase unit in this station wiped due to increased deflection of two thrust bearing-guide bearing support rings which caused the bearing pot to become eliptical and the guide bearing to be squeezed in the north-south direction. The trouble was eliminated for the present by removing shims from under the two supports. Similar realignment of the guide bearing support beams has been accomplished on other units before failure occurred.

At Holtwood Station (Pennsylvania) with 10 units, the vertical expansion or growth of concrete at two of the units installed in 1912-13 caused the floors in this section of the station to move upward in proportion to their distance from the rock foundation. This movement raised the base of the generator stator 1.5 inches above its original elevation on one side and .75 inches on the opposite side. The shifting of the thrust bearing support with respect to the water wheel resulted in the band of the 19 foot diameter runner contacting the edge of the overhanging movable guide vanes and the lifting of the brake supports with respect to the thrust bearing resulting in the brake shoes contacting the brake ring on the generator rotor. The unequal expansion or growth on the two sides resulted in the shaft's being out of plumb and the bearings out of alignment which in turn, in one unit, resulted in a shaft bend of a few thousandths of an inch.

After 27 years of operation a two-inch spacer plate was installed between the shaft coupling flanges to properly recenter the water wheel runner and the generator rotor in the vertical direction. The position of the generator stator was shifted slightly, redowled and shimmed to a horizontal position to correct the out-of-plumb condition of the shaft. A tapered shim 15 feet in diameter was placed under the thrust bearing support to restore it to horizontal position.

Vertical expansion of the concrete of the remaining eight units in this station installed in steps from 1909 to 1923 has not presented a problem.

At Buck Station (Virginia) the vertical growth of the concrete substructure appeared as a crack one inch wide around the periphery of the pit liner, peripheral cracks around the upper section of the draft tube, the breaking of many of the speed ring stay vanes and misalignment of the water wheel and generator. After 30 years of operation the movement of the parts was such that the concrete which supported the generator and water wheel was entirely replaced from the generator floor down to a level about five feet below the floor of the scroll case.

At the Falls Station (North Carolina) with three units built at the end of the dam and against a solid rock wall at the river bank, realignment has been required about every three years since the plant was placed in operation. The

unit nearest to the river has the greatest movement and the unit next to the shore has the least. Vertical concrete growth has caused the stay vanes of the water wheel speed rings to be cracked and the head cover raised and shifted with respect to the curb plate. The generator stator has also moved with respect to the water wheel speed ring. The runner clearances have been decreased on one side and the wicket gates thrown out of plumb and caused to bind. The head cover fit to the speed ring has been machined and shifted several times and the generator stator has been correspondingly shifted to correct the alignment.

At Wallenpaupack Station (Pennsylvania) after 18 years of operation two units were taken out of service primarily to install new stator windings but detailed inspection of the entire unit was made and a cracked runner, both lower seal rings and lower distributing rings were replaced. Precision checks on the water wheel speed rings and on the generator bed plates showed no evidence of either vertical or horizontal concrete growth, and checks on the shaft straightness by plumb line showed one shaft straight and the other with a bend in the center of the rotor shaft in the order of .0085 inches in 36 feet of length.

Lack of alignment data on the unit as originally erected makes positive conclusions on misalignment difficult to draw but it is believed that the lower generator bearing bracket's being out of plumb and a loose condition of the stator frame were the cause of the bent shaft and worn and cracked water wheel parts.

At American Falls Station (Idaho) growth of concrete resulted in the cracking of the downstream concrete walls of the semi-spiral water wheel casings and in the cracking of the cast iron stay vanes of some of the water wheels. Repairs to the water wheel casing walls were made by large prestressed vertical tie rods recessed in grooves cut in the face of the wall covered by a well reinforced concrete facing slab. Repairs to the water wheel stay vanes were made by fitted steel brackets studded to the cast iron vanes and speed rings, and the generator misalignment was corrected.

At E. J. West Station (New York) no appreciable growth of concrete has been experienced but misalignment of one of the two units followed the accidental connection of the unit to the line at full voltage while the unit was at standstill. The generator stator dowels were sheared, resulting in slight shift of stator and thrust bearing bracket. This misalignment was not enough to require the dismantling for several years as no contacts were made in the generator guide bearings or at the runner clearances and there was only a slight increase in wear in the water wheel lignum vitae guide bearing. At the time of general overhaul the unit was realigned and the radial dowels installed.

Recommendation

In the alignment of a unit either during the original installation or at the time of subsequent maintenance the following two factors are essential:

1. All bearing housings must be in line and their center line plumb.
2. The shaft must be straight and the rubbing surface of the thrust bearing runner must be normal to the axis of the shaft.

After insisting that the center line of the bearings must be plumb, it must be admitted that it is probably true that good operation will not be seriously affected even if the center line of the bearings at original alignment or thereafter is not actually plumb IF all guide bearings are in line throughout their length (i.e. a uniform tilting). Under this condition the guide bearings are

called upon to carry excess load, but present day designs are probably adequate to care for a considerable amount of tilt. This condition, however, is unhealthy and should be avoided initially and watched closely when subsequently discovered. We must always be wary of the erection or maintenance supervisor who takes the attitude of—"That's good enough—'settlement of the foundation' will explain everything if someday the machine grinds itself to a stop."

As stated at the outset, adequate procedures for the initial alignment of the unit are well known and not difficult although requiring care and precision,* but they depend upon the units' being dismantled in order to make the necessary observations and measurements to satisfy our two criteria. The checking of alignment following a period of operation and without dismantling the units should be possible without too much difficulty and at intervals of approximately ten years. The procedure can be greatly simplified by providing certain check points in the original design of the unit, but at some additional cost these check points may be added to a unit already in operation. These check points should include surfaces on the shaft, not subject to wear, which would be accessible without dismantling of the bearing supports, points of reference on the top surface of each bearing bracket and on the head cover of the water wheel. Points of reference for precise leveling from a permanent bench mark should be available at quarter points on the water wheel head cover, at the intermediate guide bearing when provided, and on the thrust bearing bracket. Appropriate readings taken at each of these points when the unit is first aligned and comparative readings at a later date will give both indications of loss of alignment and suggestions as to the type of corrective measure required.

Probably the most satisfactory method of obtaining these measurements is from plumb lines dropped from a point above the upper generator guide to the water wheel head cover. This is not too difficult provided that the generator rotor spider has openings between the shaft and the field yoke which is common with large low speed units. In this procedure either two or four plumb lines 90° apart are suspended from a point above the upper guide bearings, passed through the rotor openings and terminate in heavy plumb bobs in oil pots on the water wheel head cover at the lowest possible elevation. All bearings are removed with the exception of the thrust bearing and readings taken by micrometer between the plumb lines and the shaft at as many points as deemed necessary. An electrical circuit completed through the micrometer bar, the unit shaft, the plumb line, a battery and headphone provides an acceptable method of determining accurate measurements provided that a nonmagnetic metal is used for the contact tip of the micrometer to guard against false readings that might otherwise result from using the instrument in contact with a magnetized shaft. At the same time clearance measurements are made at the upper and lower water wheel seals at points approximately in line with the plumb lines and measurements are made between the reference points on the top surface of the bearing bracket and the plumb line. The measuring

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*Erection and Alignment of Vertical Water Wheel Units by R. O. Standing, Canadian Electric Association.

*Erection of Hydro Electric Turbines and Generators by A. M. Komora, ASCE Transactions.

*Adjusting Vertical Thrust Bearings by J. E. Petermann, Allis Chalmers Electrical Review

*Mechanical Alignment of Vertical Shaft Hydro Electric Units as Practiced by TVA by C. L. Norris, A.I.E.E. Transactions

points on the shaft for each reading must be determined by a straight edge and center head. The measurements obtained can be plotted and will not only check the plumbness of the axis, but also bring to light a kink or bend in the shaft and any movement of the bearing brackets on their supports. Independent of the plumb line measurements, but a very pertinent part of overall check of the unit, observations should be made of the elevation of the water wheel runner at both top and bottom seal at points in line with the plumb lines, and accurate levels should be taken at the four reference points on the water wheel head cover and on the bearing brackets. At the same time measurement of generator air gap should be made.

This procedure does not definitely check the squareness of the thrust runner with the shaft, without the rotation of the shaft through 180° and repeating the measurements. When the shaft is rotated a very accurate measure of the angle of rotation can easily be obtained by measuring along the edge of the brake ring between a fixed mark on the ring and a fixed mark on one of the brake shoes. On units having the thrust block forged integral with the shaft, this rotation test subsequent to erection is probably not warranted; but on units having thrust block keyed to the shaft, the test may be very enlightening, showing up wear or working on the thrust block surface or bore, or non-uniformity in the thickness of the insulation between thrust block and thrust bearing runner plate.

Shop checks of combined water wheel and generator shaft are such that we have a right to expect the original alignment in the field to have small tolerance. Considering methods and accuracy of taking subsequent field checks, it would seem that tolerance in "run out" in the order of 0.004 L/D would provide satisfactory operation, but measurements beyond this value should be considered in the light of the cost of correction. In the above ratio L equals the distance from the surface of the thrust bearing runner to the point of measurement and D equals the outside diameter of thrust bearing runner.

After all the procedures for making an accurate check have been determined and provided for, the success of the job will depend not only on the accuracy with which the measurements are taken, but also upon the form in which they are recorded for a reasonable interpretation and, last but not least, for filing for future comparisons and possible RE-INTERPRETATION. To this end a proposed form of record has been set up which of course can be only a guide or a suggestion, for there SHOULD be almost as many types of record as there are types of machines and men to supervise their overhaul and maintenance.

ACKNOWLEDGMENT

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As indicated previously, this paper has attempted to assemble American operating experience and in doing so, the author is greatly indebted to many engineers and operators in many companies and especially the following:

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STATION ____________________

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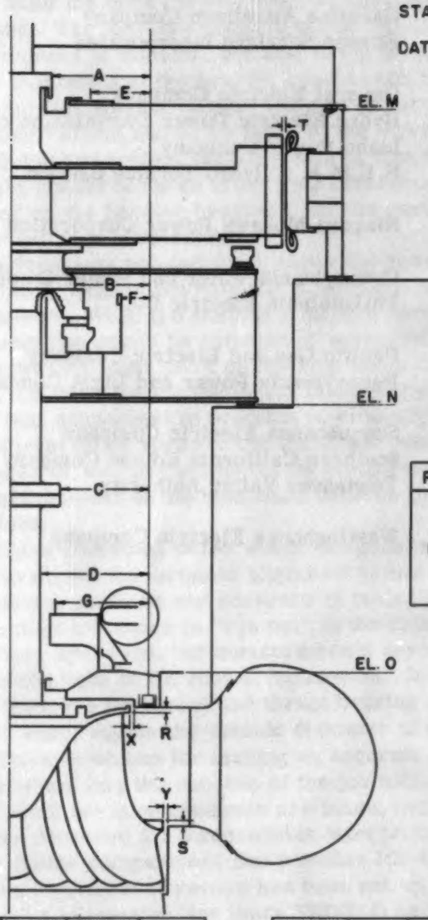

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HYDRAULIC DESIGN OF THE SANDOW PUMPING PLANT

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SYNOPSIS

The Sandow Pumping Plant is a very large vertical turbine pump installation which provides make-up water for a cooling pond serving the circulating water system of a 300 mw power plant near Rockdale, Texas. This pumping facility is unusual in its combination of large vertical pumps, long and irregular pipeline profile and surge and flow control problems. This paper describes the hydraulic features of the plant and briefly discusses the solutions to some of the unusual design problems.

INTRODUCTION

In 1951 the Aluminum Company of America started construction of a major aluminum smelting plant near the town of Rockdale, Texas, approximately 175 miles south of Dallas. To meet the extensive power requirements of this enterprise the 3 unit, 300 mw Sandow Power Plant was built for Alcoa by the Texas Power & Light Company.

Steam electric power plants require a large and dependable supply of water for condenser cooling purposes. Over 240,000 gallons per minute continuous flow is pumped through the 3 condensers at Sandow. This is cooled and used again, with a relatively small part lost through evaporation, seepage from the storage pond, and nonrecoverable plant uses. This loss must be replaced. Study of possible surface and subsurface supplies indicated that the best solution to the water supply problem was to pump river water through 12-1/2 miles of conduit to a 700 acre storage pond to be located in the plant area.

The result was the design and construction of a pumping station and pipeline, which in size and design features is unusual among water pumping facilities, particularly for a steam electric station.

Figure 1 is a general plan of the area showing the pipeline, cooling ponds and industrial plant sites.

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3. Resident Eng., Sandow Power Plant, Aluminum Co. of America.

Primary Physical Features

The pumps are mounted on an outdoor type intake structure on the banks of the Little River just below its confluence with the San Gabriel River near Minerva. Three 5-stage vertical turbine pumps furnished by Peerless Pump Co supply a rated flow of 31,500 gpm (70 cfs) discharging at 555 feet total dynamic head. The pumps are 70 feet long from bell to motor base and are driven by 2000 Hp Elliott Co motors. One of the three motors is designed to operate at 2 speeds. Figure 2 shows a sectional view of the intake structure and Figure 3 a plan of the operating deck.

The water is pumped from the intake at Little River through 11-1/2 miles of 36" concrete cylinder pipe to a small concrete regulating reservoir at the top of a hill overlooking the power plant. From there it flows by gravity through about 1 mile of 36" and 42" reinforced concrete pipe to the storage pond. The pressure section of the pipeline passes over a number of hills which make it very irregular in profile and particularly subject to water hammer, air binding and silting. The profile is shown on Figure 4.

Operating power is transmitted directly from the Sandow Power Plant through a 15 mile 33 kv overhead transmission line.

The Water Supply Problem

Water in Texas is a scarce and valuable commodity, the allocation of which is carefully controlled. Although a supply for necessary makeup was available in the Little River during high flow periods, much of it was dedicated for other uses, and frequently there was little or no flow at all. An extensive hydrologic study indicated that with adequate storage capacity enough water could be pumped from the river during high flow periods to satisfy power plant requirements and still provide for all prior rights. Pumping is generally restricted to the non-irrigation months from September 15 to May 15 and further restricted to pass an established minimum river flow at all times.

The hydrologic study brought out the possibility that there may be long periods of flow not far above the established minimum. Since a single pump moves over 38 cfs unless throttled it was decided to provide one 2 speed pump which could operate if necessary at a reduced speed and economical power consumption for an extended period. One of the three pumps was therefore equipped with a 2 speed motor which permits operation at 900 rpm, 2000 hp or 600 rpm, 700 hp.

Depending on the water required and the water available, the pumping rate can be varied from one pump at low speed to three pumps, an operating condition which would be set only in extreme emergencies. It was expected that make up requirements in the cooling pond during normal years would require only single pump operation, and the pipe size economic study was based on this assumption. One pump provides approximately 55% of maximum capacity, 2 pumps 85%. The addition of the third pump will contribute only 15% to the maximum possible output. This is a result of the rapid increase in frictional resistance in the pipeline as flow increases (351 feet of the design head of 555 feet represents pipe friction, 204 feet static lift). The third pump also serves as a standby in the event that one of the pumps is out of service.

Flow Control

Size alone was not a major problem, but severe water hammer possibilities and certain special control requirements greatly increased the difficulties in providing for safe and economical pumping plant operation.

One of the important basic considerations in the design of all controls was the requirements that this large and relatively complex pumping station must be safely operated without the attendance of an operator. Therefore, the final design requires that an operator be present only when the pumps are started up. It is also desirable to shut down the pumps at the pump station, but in emergencies they can be tripped by cutting power at the control room of the Sandow Power Plant.

The primary control problem was to protect the pumps and pipeline against the particularly severe water hammer surges which design studies indicated might occur following improper startup or shutdown of the pumps, and particularly following emergency pump tripout. The nature of these surges and the design of surge control are of special interest and are discussed in detail below.

A secondary problem was the pump manufacturer's requirement that reverse flow not be permitted through the pumps. It was not practical to protect the pumps with a common check valve because of water hammer considerations and the necessity for careful flow control in starting and stopping. Each pump was therefore equipped with a 350 psig - 20" S Morgan Smith Co Rotovalve (rotating plug type) operated by oil pressure from an oil accumulator system. A special handwheel controlled throttling device was built into each valve to permit reduction of pump flow during occasional periods when the intake water level is too low for proper impeller submergence. A gate valve was mounted downstream from each Rotovalve to permit servicing and testing of the pump discharge valve system.

The other major protective features incorporated in the design of the station controls are as follows:

a. Oil pressure switch to trip pumps if the oil pressure in the pump discharge valve operating system falls below a given value. This control is set to insure that there will be enough pressure to close all three valves. There is always the possibility that the oil pump or compressor may fail or that the oil tank or lines may leak.

b. Pipe line pressure switch to trip pumps if the pump discharge pressure falls below the minimum required for satisfactory and safe pump performance. This protects the pumps during seasonal refilling of the line and also in the event of a pipeline break.

c. Intake water level switches to trip the pumps when the water level in any pump intake chamber falls below the minimum required for proper submergence (due, for example, to clogged screens).

d. Time delay relay to prevent the restart of pumps after tripout until a fixed time interval has passed. This control permits dissipation of the severe water hammer surges following emergency tripouts.

e. Overcurrent relay to trip motors on overcurrent, short circuit or in the event of locked pump motor.

f. Motor bearing temperature detector relays to trip motors with overheated bearings.

In every case the tripout circuits shut down all operating pumps regardless of which pump may be in difficulty. This is to prevent the possible tripout of

pumps in close succession with a resulting undesirable buildup of surge waves. The pump discharge valves, which are solenoid controlled, close immediately on the "emergency closure" cycle when power to the pumps or to the valve controls is cut off. The total time of closing is from 5 to 8 seconds on this cycle.

Water Hammer and Its Control

In the design particular care was directed to the determination and control of pressure surges or water hammer. Theoretical analysis indicated that following a sudden pump tripout and the rapid deceleration of the pumps the column of water in the pipeline would, within a few seconds, break up into several individual sections as subnormal pressure waves brought the operating head to vapor pressure. This is the surge phenomenon of water column separation in an extreme form. The subsequent uncontrolled rejoining of these various sections would create excessive and possibly damaging pressures throughout the pipeline.

Figure 5 shows an actual test record which clearly illustrates this phenomenon. This record was taken at Valve House No. 8, about 7000 feet from the pipeline discharge. See Figure 4. No surge protection devices were in operation and flow prior to final valve closure has been reduced to 3 feet per second (design maximum 10 fps) to insure that surge pressures would not be excessive. Final valve closure took 80 seconds as compared with the 8 seconds to be expected for complete pump stoppage and valve closure following emergency tripout. The severity of the surge regime is immediately apparent. Figure 5 does not represent the highest pipeline surges reached during this particular test, but does most clearly illustrate the nature of water column separation and rejoining.

The severity of the pressure drop following interruption in the normal flow regime actually made water hammer control relatively simple. A paradox indeed! Following a pump tripout of even a single pump at low speed the presure through the pipe line fell below atmospheric, and at all high points to vapor pressure. In fact, in order to prevent the occurrence of sub-atmospheric transient pressures in the pipeline during normal shutdowns pump discharge valve closure time had to be set at 6 minutes.

The primary requirements for surge control were, first, to prevent the formation of high vacuums following tripout, and secondly, to prevent the abrupt rejoining of broken water column sections with the resulting sudden destruction of flow velocity. A controlled closing vacuum breaker serves both these purposes. This device, a simplification of the familiar "surge suppressor", requires no outside source of power to open; vacuum condition in the pipeline provides the necessary force to open the valve, and a small oil dashpot controls the rate of valve closing to permit water discharge and gradual deceleration of the water columns. This closing time was set from 3 to 5 minutes for each valve. Nineteen 4 inch Simplex Controlled Closing Air Valves were mounted along the line, 3 on the pump discharge header and 2 at each of 8 valve houses at major pipe line summits. The discharge from each set of valves was carried by pipes or ditches to nearby stream beds. Had the surge expectations been less severe and the probability of vacuum less certain, the surge valves would have required outside sources of power to function, thus adding more controls and 15 miles of low voltage transmission line to the cost of surge protection.

Surges resulting from pump startup, the only other major cause of water hammer, were controlled by starting the pumps against closed discharge valves, then opening the valves slowly at a set uniform rate of about 4 minutes. The shutoff head of the pumps is 730 feet.

Field Tests

The pumping plant went into operation in November 1952 with one of the ultimate 3 pumps in service. Extensive single pump hydraulic tests were conducted in January of 1953.

The test program was set up to check all phases of the plant operation; however, the majority of the tests were directed at determination of the surge regime and the setting of surge relief valves and safety devices. The long length of line and relative inaccessibility of many of the valve stations required considerable care and planning to give the desired results with a minimum of time and expense. Three radio cars, portable generators, recording and sight gages and about 25 men were stationed at the pump and along the pipeline during a program consisting of 4 test tripouts (communication between the Sandow Power Plant and the pump station was normally carried on by radio).

Figure 5 discussed above, is one of the test records from the first test. The subsequent 3 tests were conducted with increased flows and all vacuum breakers in operation. The final test was a full emergency tripout of one pump (5.5 fps line valocity and 8 second valve closure). The vacuum breakers, even though not fully adjusted, reduced the surges to negligible values throughout the pipe line.

The test results permitted the fixing of the 4 and 6 minute valve opening and closing times noted above. The long closing time is required to keep the vacuum breakers shut except in emergencies. This avoids the considerable loss of water which normally occurs during the 3 to 5 minute breaker closing time.

Additional Plant Features

The pumping station, including pumps, valves, 20 ton station crane, switchgear and transformers were all designed to be outside and exposed to the elements. A small enclosure on one side of the operating deck provides shelter for operating personnel, the radio, the oil accumulator system, flow meter box and some minor electrical equipment. All equipment was protected against a minimum ambient temperature of minus 10°F and 30 mph wind.

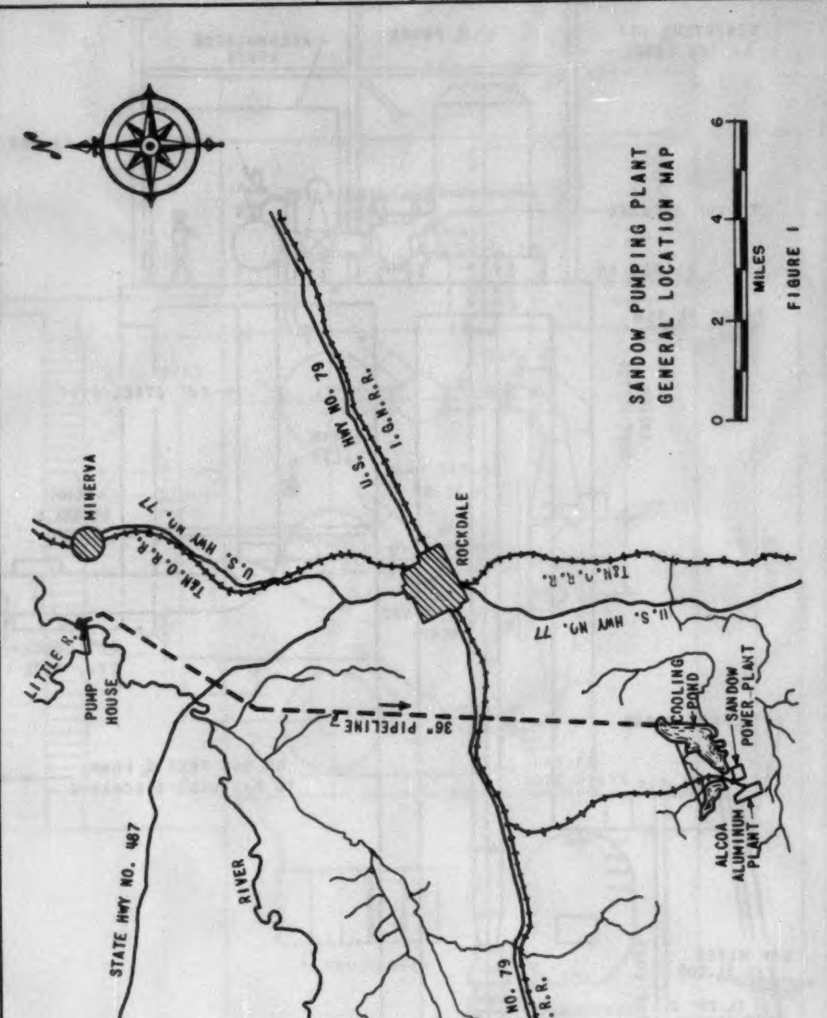
The deck is 53 feet from the low water river surface and 20 feet above grade. This high deck was required to pass floods 50 feet above normal low water since flash floods of extreme severity are common on this normally small and placid river.

Pipeline flow is continuously recorded on an integrating recording meter with flow measured through an orifice plate placed in a horizontal section of the pipeline about 100 feet upstream from the pumping station.

Air accumulations in the pipe line are handled by 16 automatic air release valves at high points. 8 additional connections have been provided for future air release valves if required. No air binding difficulties were observed during the tests.

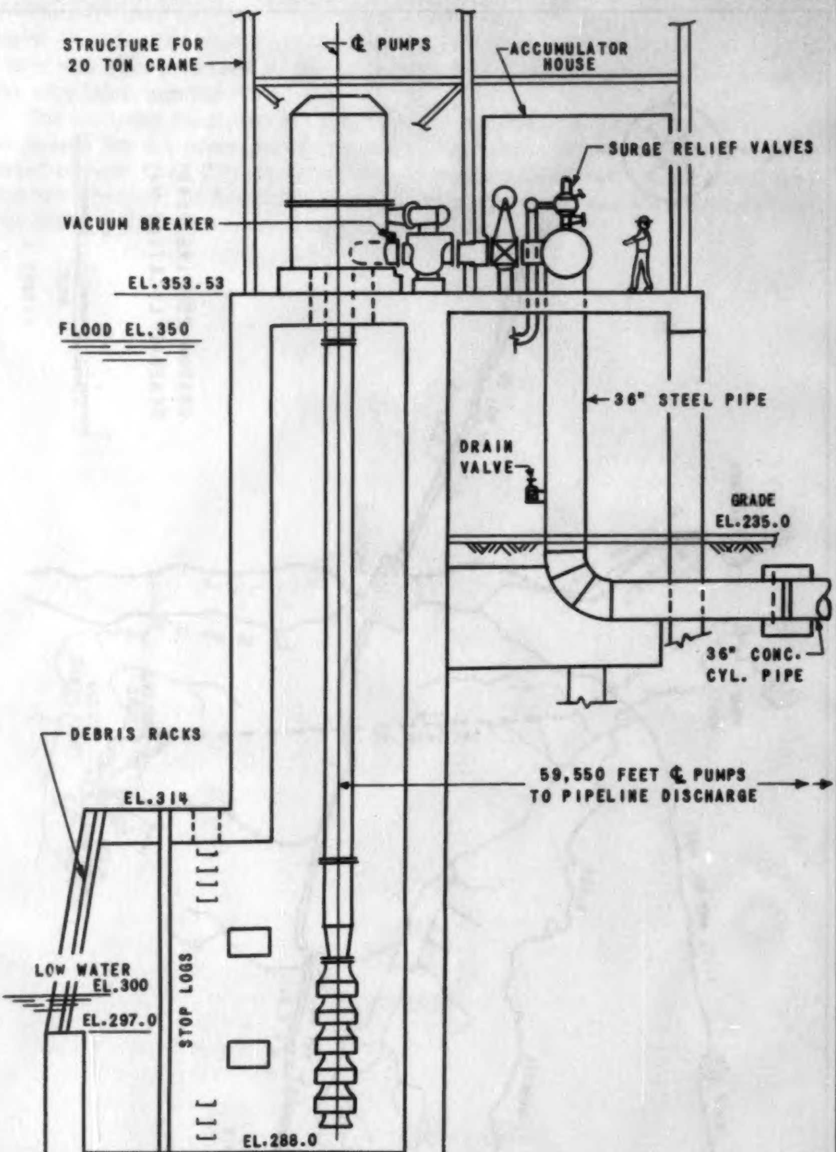
Four 6" blow off valves and 16 additional blanked off blow off connections were provided for sand and silt removal and draining of the line. An 8" drain valve was also provided at the pump station to permit convenient draining of the pipe back into the river.

The pumping plant was built for the Texas Power & Light Company, acting as agents for the Aluminum Company of America. Ebasco Services Incorporated of New York City designed the plant and supervised construction. H. B. Zachry Company of San Antonio layed the pipe, which was manufactured by the Gifford-Hill-American Company of Dallas.



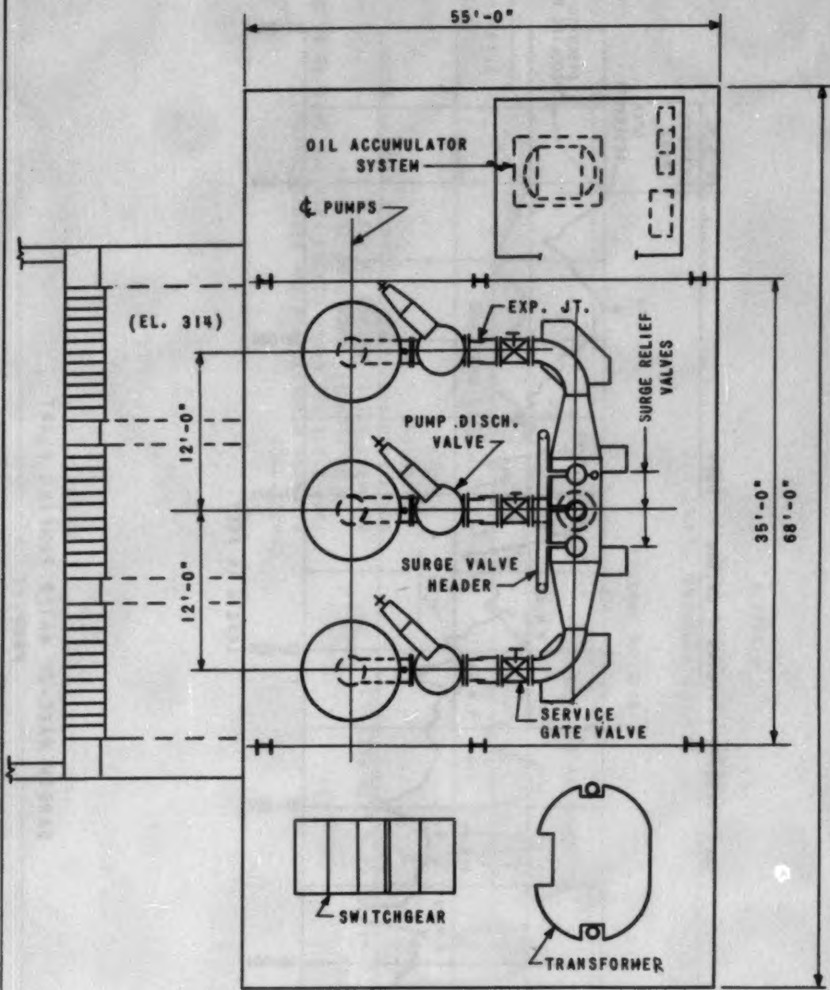
SANDOW PUMPING PLANT
GENERAL LOCATION MAP

FIGURE 1



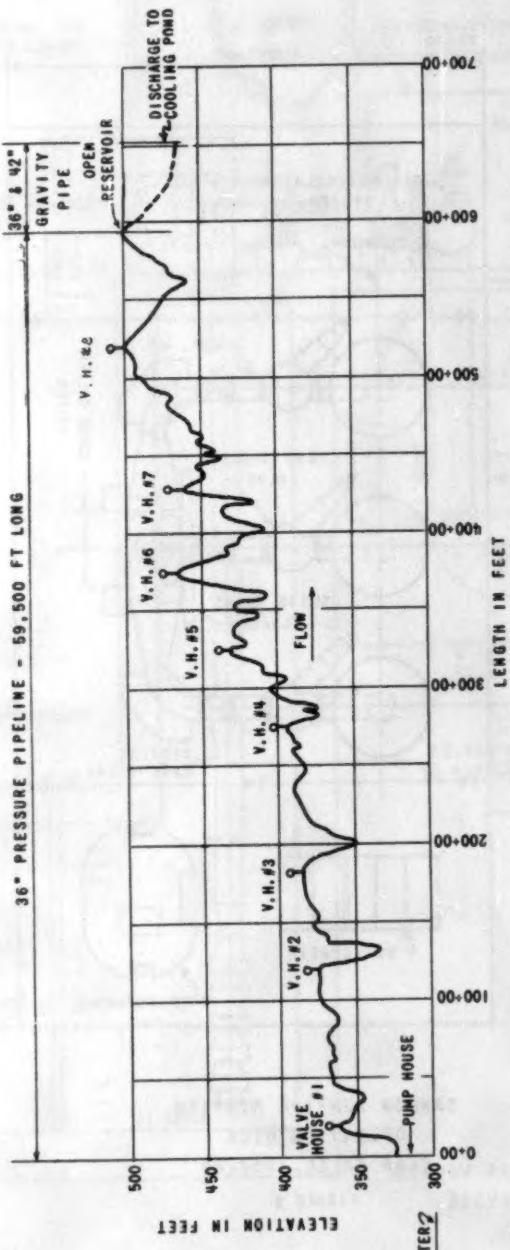
SANDOW PUMPING STATION
SECTIONAL VIEW

FIGURE 2



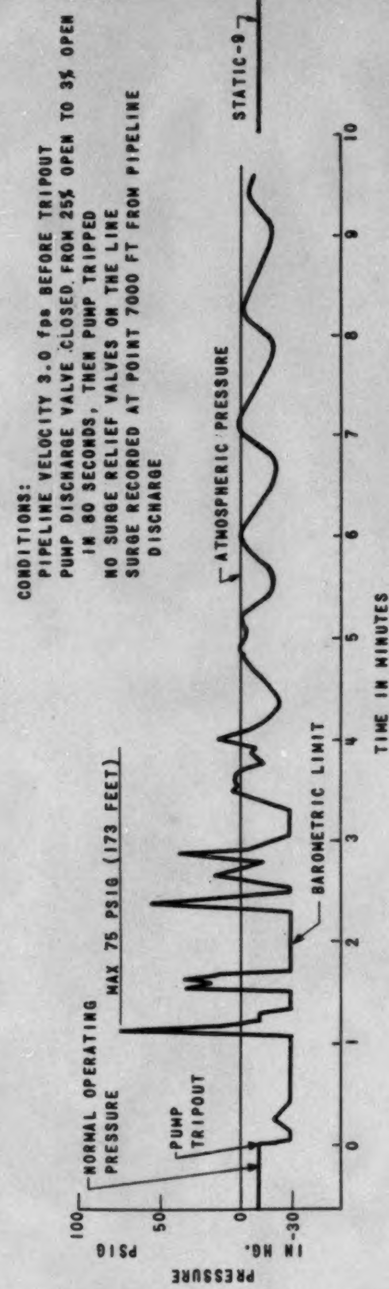
SANDOW PUMPING STATION
OPERATING DECK
PLAN AT EL. 353.33

FIGURE 3



SANDOW MAKE-UP WATER PUMPING PLANT
PROFILE

FIGURE 4



SANDOW PUMPING PLANT
SURGE FOLLOWING PUMP TRIPOUT

FIGURE 5

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STEEL LININGS FOR PRESSURE SHAFTS IN SOLID ROCK

E. W. Vaughan,* M. ASCE

SYNOPSIS

The design of steel linings for pressure shafts for underground hydro-electric plants is concerned mainly with the problem of proportioning the steel to resist both the internal pressure under operating conditions and the external ground-water pressure when the shaft is empty. The methods used in the design of two such shafts in Brazil are discussed in detail and their general features compared with those of other shafts constructed elsewhere. A general approach to the problem is presented, together with pertinent details of design and construction.

INTRODUCTION

Steel lined pressure shafts in solid rock are used in lieu of penstocks to supply underground hydro-electric plants and, occasionally, to supply conventional above-ground plants. Where conditions are favorable to underground construction, such shafts are often more economical than penstocks. This is due to the effective use of the rock itself in resisting internal pressure thus allowing a considerable reduction in the amount of steel required. The design of such shafts is therefore concerned principally with the proportioning and detailing of the steel lining. The ensuing discussion will be confined to this subject, other matters being touched upon where relevant but only to the extent necessary to an understanding of the factors to be considered in the design of the steel.

The purpose of the steel lining is two-fold: to provide an impervious membrane and to sustain a sufficient amount of the internal pressure to prevent over-loading of the surrounding rock. The steel lining must carry its load without excessive yielding or rupture and must be stable under the external pressure of the ground water when the shaft is empty. The methods for proportioning the steel to resist both conditions of loading have varied for different projects,the art progressing with the accumulation of experience.

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These methods include both empirical and theoretical analyses, the results of which are modified in accordance with the judgment of the engineer to take into account the indeterminate and variable factors.

Since the problem is highly complex and involves to a considerable degree the proper interpretation and evaluation of geologic conditions at the site, the factor of judgment is predominant and the necessity for expert geologic opinion is obvious. It is hardly possible to derive a standard method for the design of pressure shaft linings as each has its own individuality and must be treated accordingly. On the other hand, the same basic principles are involved in all cases. It is the purpose of this paper to present and discuss those principles, first by describing what was done on two pressure shafts constructed in Brazil, then comparing these with others constructed elsewhere, discussing the differences and outlining a rational general approach to the problem.

Nilo Pecanha Pressure Shaft

Description Figure 1 illustrates the pressure shaft for the Nilo Pecanha Underground Plant[1] (formerly known as Forcacava) of the Rio de Janeiro Tramway, Light and Power Company, near the city of Rio de Janeiro, Brazil. This shaft is approximately 20 feet in inside diameter and 1950 feet long terminating in a distribution manifold with six branches, 5'-3" in inside diameter, having an aggregate length of about 876 feet. The steel lining of the shaft proper varies from 9/16" to 1-1/8" in thickness while that of the manifold varies between 3/4" and 1-3/4" in thickness. The maximum static head is 1123 feet and the maximum operating head, including water hammer, is 1287 feet.

The shaft is constructed in hard gneiss of generally good quality. Locally the rock is kaolinized in varying degree, the kaolinization occurring in bands ranging in thickness from a fraction of an inch to three feet or more. The material in these bands varies from a relatively hard rock to a soft, clay-like material. Scattered bands of this altered rock occur throughout the major portion of the shaft, but only in two zones is the kaolinization sufficiently extensive to influence design of the lining. The lower portion of the shaft also passes through a diabase dike, which, though typically jointed, is sound.

The lining is made of carbon steel plates of high tensile strength. The plate thickness varies in accordance with rock conditions and internal pressure. In the upper kaolinized zone which crosses the inclined section of the shaft at about mid-height, the maximum thickness is 1-1/8". In the lower kaolinized zone near the power cavern the rock is of still poorer quality, the internal pressure is greater and the tunnel is located close to the large excavation for the power cavern. Because of these conditions, the strength of steel lining required is greater than that needed in the upper kaolinized zone. This additional strength is provided by reducing the diameter of the shaft from 20 feet to 15'-7" and increasing the plate thickness within the zone to 1-1/4". In addition, the concrete filling between the rock and the steel lining is heavily reinforced.

Drainage facilities are provided to relieve external groundwater pressure which might otherwise cause collapse of the lining during future periods of unwatering. These facilities consist of a 12" steel pipe embedded in the shaft invert and tapped at intervals by short, drilled holes. The drilled holes act as lateral drains to relieve pressure at the contact surface between the steel and the concrete envelope.

Note: Raised numbers refer to the list of References on pages 23 and 24.

U-shaped anchors of reinforcing bar, welded to the outside of the steel lining and embedded in the concrete envelope, are provided to increase the stiffness of the lining.

Construction Construction of the pressure shaft followed procedures which are more or less standard for this type of work. Excavation of the lower section, downstream from the bend, was advanced as a full heading tunnel, starting from the power cavern and progressing in an upstream direction. The sloping portion of the shaft was constructed in two operations. A pilot raise was excavated at invert level from the lower bend to the previously excavated valve chamber at the upper end of the shaft. The roof was then slashed out to full size, starting from the top and working downward, the pilot raise serving as a chute for spoil.

Steel lining components were received in the field as formed plates with edges prepared for welding. These were assembled, welded, stress-relieved and radiographically inspected in two fabricating yards at the job site. The upper yard, located near the portal to the valve chamber access tunnel, was used for assembly of all pressure shaft sections. The lower yard, near the power cavern access tunnel, was used for assembly of components for the distribution manifold.

In the upper yard, the formed plates were assembled into cylinders 20 feet long, each cylinder being made up of 7 plates joined by longitudinal butt-welded seams. Finished sections were transported by tractor and trailer to a work station located in the valve chamber. Here they were joined in pairs to form 40 foot lengths. At the same time the anchors and other accessories were attached. The 40 foot sections were lowered into their proper positions in the shaft by means of a cable-operated erection car which ran on rails placed in the invert. After each section was lowered into the shaft and properly positioned, it was welded to the previously placed section. The car was then pulled back up the shaft and positioned to receive the next section of lining. As each part was installed, the space between the steel and the rock was filled with concrete, which was placed by pump in the flat portion of the shaft and by chute in the inclined portion.

At the same time that these operations were proceeding in the pressure shaft, similar assembly operations were being executed in the lower yard. The distribution manifold components assembled in that location were transported to the power cavern by tractor and trailer. There they were unloaded by travelling crane, lowered into the machine pit area, skidded into final position, and welded together. As each section or two was placed it was concreted in, the concrete being placed by pump.

Concreting was followed by pressure grouting to insure tight contact between rock and concrete and between concrete and steel. This was done in three stages. The first stage consisted of low pressure grouting of the contact between concrete and steel. This was followed by intermediate pressure grouting of the concrete to rock contact and, later, by high pressure consolidation grouting of rock immediately adjacent to the tunnel.

After all concreting and grouting was completed, vertical holes were drilled through the steel lining and concrete into the drain pipe shown in Figure 1. This drain pipe was originally installed with tight joints to prevent its being plugged by concrete or grout and the drilled holes were necessary to make it effective as a relief drain. After each drill hole was completed, the hole in the steel liner was tapped and plugged with a steel plug. The projecting end of the plug was then ground off flush with the inside of the lining.

The completed lining was painted inside with coal-tar enamel for protection against corrosion.

Design The design of the steel to resist internal pressure was based upon the assumption that the steel, rock and concrete would deform alike when loaded. Thus, any tensile stress in the steel caused by internal pressure would result in an increase in diameter which would be accompanied by compressive deformation and consequent compressive stress in the surrounding concrete and rock, provided the three materials were in intimate contact at all times. Since contact between the three materials is likely to be somewhat less than perfect, due to the probability of cooling of the steel after installation and, since progressive plastic deformation of the rock might occur under sustained loading, it was evident that the elastic deformation of the steel could be somewhat greater than the non-plastic deformation of the concrete and rock. To take this probability into account, it was estimated that the radius of the steel lining would increase 0.04 inches before any pressure could be transferred to the concrete and rock. In effect this amounted to assigning a fixed pressure to be carried by the steel without any assistance from the surrounding material. The remaining pressure was then distributed between the steel and the concrete and rock in proportion to their load-deformation characteristics.

The physical properties of the rock at Nilo Pecanha were not well known. Laboratory tests of two cores of sound gneiss showed that the material had a crushing strength of 5700 psi and an average modulus of elasticity in compression of 5,250,000 psi. No tests were made of the badly kaolinized material as it was impossible to obtain any cores. In-situ tests would have been desirable. However, due to the extreme urgency of the construction program, none were made.

Considerable basic information was available from a study of successful European underground plants, a part of which consisted of laboratory and field tests of rock similar to that at Nilo Pecanha. By comparison and evaluation of the existing data, it was concluded that the "effective" modulus of elasticity of the sound gneiss could be taken as 1,400,000 psi, which is considerably less than the modulus of elasticity determined by laboratory tests of diamond drill cores. For the kaolinized material an "effective" modulus of elasticity of 280,000 psi was assumed. It should be noted here and will be discussed later, that the "effective" modulus of elasticity is in reality a modulus of deformation, modified in accordance with an assumed value of Poisson's ratio.

Using the "effective" moduli of elasticity of the rock, calculations were made to determine the thickness of lining required in each type of rock and for steels of different strengths. Several published methods were available, all based on the elastic theory but providing different means for taking its limitations into account. These methods, together with some original attacks, resulted in a variety of answers. By elimination of those which required extreme or unreasonable assumptions, the remainder were found to be in fairly good agreement.

These preliminary studies indicated the advisability of using a relatively high strength steel in order to keep the maximum plate thickness within acceptable limits for good welding and, at the same time, reduce both welding and shipping costs. The advantages of the thinner plate were felt to outweigh the disadvantage of a somewhat higher price for the raw steel. The steel selected was ASTM-A212 Grade B, firebox quality, modified, which has a minimum yield point of 42,000 psi for plates one inch or less in thickness.

The allowable steel stress was established at 80% of the yield point for the pressure shaft and 70% of the yield point for the distribution manifold where a higher safety factor was desired.

Once the type of steel and the allowable stresses were established, the plate thicknesses required in sound and kaolinized rock were determined using the most reasonable of the methods investigated in the preliminary study. The differences were rationalized and resulted in proportioning the steel to take 30% of the internal pressure in the sound gneiss in the lower portion of the shaft and 67% of the internal pressure in the kaolinized material. The minimum thickness of lining was set at 9/16" to provide sufficient stiffness for handling and placement. The lining for the downstream ends of the manifold branches, which project into the power cavern, was designed for the full internal pressure. The internal pressure used for design included the effects of water hammer in all cases. With the thicknesses at key points determined in this way, the proportioning of the intervening sections consisted of providing reasonable transitions in thickness between those points.

The lining design was completed and all steel plate ordered before the shaft was excavated. The excavation proved the effectiveness and accuracy of the geologic investigations and interpretation as only one change was required in the original design. The lower kaolinized zone shown in Figure 1 proved to be far weaker than anticipated. The original design was altered therefore by reducing the diameter of the lining and increasing the plate thickness in the downstream portion of the pressure shaft and by reinforcing the concrete envelope throughout the affected length.

The effects of external pressure were not taken into account in determining the thickness of the steel. Instead, it was judged better to provide relief drainage to prevent external water pressure from attaining damaging magnitude. This decision resulted in lining thicknesses considerably less than would have been possible if the steel had been designed to carry the probable external pressure.

Performance The pressure shaft was filled in September 1953 and has been in continuous operation since that date. It has been performing satisfactorily although it has not yet been subjected to the maximum pressure assumed for design. The drainage system is evidently operating as anticipated and observations of its discharge indicate that the steel lining is water-tight throughout.

Cubatao Pressure Shaft

Description The Cubatao pressure shaft of the Sao Paulo Light and Power Company, Limited, near Sao Paulo, Brazil, is illustrated in Figure 2. This shaft is 10'-8" in inside diameter for an approximate length of 4700 feet of which all but the uppermost 238 feet is lined with steel. A transition section 9'-10-1/8" in inside diameter and 230 feet long connects the shaft with the distribution manifold, which varies from 9'-10-1/8" to 5'-3" in inside diameter and has an aggregate length of 540 feet, including the turbine branches. The steel lining varies from 1/2" to 29/32" in thickness in the shaft proper. In the transition section, the thickness increases from 1" to 1-15/16". Within the distribution manifold plate thicknesses vary from 1" to 2". Maximum static head on the shaft is 2357 feet while the maximum operating head, including water hammer, is 2709 feet.

The shaft is located entirely within granitic gneiss of excellent quality

containing no altered material such as was found at Nilo Pecanha.

The lining is made of two different types of high tensile steel, ASTM-A-299 being used for the pressure shaft and ASTM-A-225, modified, being used for the transition and distribution manifold. These steels have minimum yield points of 42,000 psi and 52,000 psi, respectively, for plate one inch or less in thickness. Because of the uniformity of the rock throughout the entire length of the shaft, the thickness of plate increases continuously from the upstream to the downstream end, in accordance with the increase in internal pressure.

No drainage facilities nor stiffening devices are provided.

Construction Construction of the pressure shaft is now under way, following procedures similar to those used at Nilo Pecanha. Lining components for the pressure shaft, fabricated as curved segmental plates will be assembled in the field, four such plates being required for each 20 foot length of lining. Lining sections for the transition and distribution manifold were shop-fabricated as full-circle pipes in lengths up to 20 feet. All connections will be butt-welded, stress-relieved and radiographically inspected.

Placement of the lining, concreting, grouting and enamelling will be accomplished in the same manner as at Nilo Pecanha.

Since the steel lining begins a considerable distance downstream from the intake, provisions were necessary to prevent water from the intake and connecting tunnel leaking past the lining and filling voids in the rock or concrete in such a manner that dangerous pressures could develop against the external surface of the steel. These provisions consist of a cut-off ring of 6" x 6" x 3/4" angle, welded to the liner near its upstream edge and embedded in the concrete backing, as well as pressure grouting of the concrete and rock to form a cutoff.

Design Design of the steel lining for the Cubatao shaft was accomplished in a simpler and more direct manner than was possible at Nilo Pecanha. This was the result of more uniform rock conditions, actual measurement of the load-deformation characteristics of the rock, development of a method for calculating the resistance of the lining to external pressure and practical experience gained at Nilo Pecanha.

Deformation tests of the rock were made in a horizontal adit about 6-1/2 feet in diameter and approximately 160 feet long. The end of this adit was sealed off by means of a watertight bulkhead to form a test chamber approximately 30 feet long. This was filled with water and the pressure increased in increments to a maximum of 500 psi. Deformation was measured by gages on vertical and horizontal diameters at two cross-sections approximately 3-1/2 feet apart. For a pressure of 500 psi the deformation modulus, which is the ratio between unit pressure and unit elongation of the diameter, was found to be 1,600,000 psi for the vertical diameter and 3,200,000 psi for the horizontal diameter. The "effective" moduli of elasticity corresponding to these values were calculated at 1,800,000 psi and 3,900,000 psi, respectively. To keep the design on a conservative basis, the lower value was used.

Laboratory tests of core specimens showed moduli of elasticity ranging from about 5,000,000 psi to 10,000,000 psi and compressive strength of about 12,000 psi.

A second series of tests was made in the same chamber to determine the proportion of load that would be carried by the steel lining. A length of steel pipe 43-1/4" in inside diameter with 1/2" wall thickness was inserted in the test chamber and concreted in place. Water was introduced and the pressure built up in increments as before. Diametral deformation was measured on

each of 4 diameters as in the original series of tests. These tests showed that up to a pressure of 710 psi (1640 feet of water), the steel deformed just as if it were in the open without any rock or concrete to back it up. Evidently there was sufficient space between the steel and the concrete to allow freedom of deformation of the steel under this load without any transfer of pressure to the surrounding material. Whether the lack of contact was due to thermal shrinkage of the steel or to a poor concrete filling job or both is not known. It is possible that plastic deformation of the rock and concrete could have played a part also. However, it is doubtful that such plastic deformation could have been very important as the loading was not maintained for a great length of time. In any event, the tests emphasized that for design purposes a reasonable amount of expansion in the steel should be considered to take place before the surrounding material receives any load whatever. In addition the results pointed to the desirability of obtaining good contact by proper concrete placement followed by grouting and indicated that efforts to obtain good contact should not be spared.

The allowable stress for the steel lining of the shaft was taken at 30,000 psi or about 70% of the minimum specified yield point. At this stress the elastic expansion of the radius of the lining was calculated at 0.064". Estimating that approximately one-half of this deformation would be accounted for by thermal shrinkage of the steel and possible lack of tight initial contact between the steel and concrete, the remaining deformation was considered equal to the radial compression of the rock and concrete envelope.

This deformation was proportioned between the rock and concrete in accordance with their modulus of deformation and modulus of elasticity, respectively, assuming that transfer of stress through the concrete would be a case of direct radial compression with no tangential stress. It was found that 760 psi would have to be absorbed by the concrete and rock if the steel lining were stressed to 30,000 psi, irrespective of the magnitude of the total internal pressure. Any change in the pressure applied to the rock and concrete would, of necessity, be accompanied by a similar change in the steel stress, provided the steel were not stressed beyond the elastic limit. The problem of designing the steel to resist internal pressure was thus considerably simplified. It was only necessary to determine what thickness of plate would be required to carry a pressure 760 psi less than the design pressure without exceeding a stress of 30,000 psi.

The distribution manifold was designed to take the full internal pressure, including water hammer, at a stress not to exceed 0.75 of the minimum specified yield point except for the downstream ends of the branches where the allowable stress was reduced to 0.70 of the yield point.

The transition section was simply proportioned to provide an easy transition from the calculated thickness of the downstream end of the pressure shaft lining to the greater thickness at the upstream end of the lining for the distribution manifold.

In regard to the resistance of the lining to external pressure, the original design for Cubatao followed the same idea as Nilo Pecanha. It was intended to provide relief drainage to prevent the accretion of damaging external pressures rather than to provide sufficient thickness of plate to resist those pressures safely. However, adverse experiences during the construction of the Nilo Pecanha shaft led to a reappraisal of the situation. The drain pipe had been plugged with rubbish at least once and inadvertently grouted shut for a considerable distance on another occasion. In addition, the steel lining had

buckled locally in several places due to grouting pressure. These accidents naturally led to an aversion to drains and thin plate linings. The decision was made, therefore, to eliminate the drain and to increase the thickness of lining where necessary to provide resistance against buckling under an external pressure equivalent to a water column extending from the shaft axis vertically to the ground surface. The thickness of lining required to withstand this pressure was calculated by an approximate method which showed that the entire lower portion of the lining was too thin. Thicknesses in this reach were therefore increased.

Another method of calculation was developed by the author, taking into account certain factors which were ignored in the first method. Solution by this method indicated that the thickened lining had no more than about 2/3 of the resistance to external pressure which was found by the first calculation. The pros and cons of the two methods were debated at considerable length and it was rather generally concluded that the lining was either unsafe or of dubious safety insofar as its resistance to external pressure was concerned. This brought the matter of relief drainage back into consideration but, because of the experience with the drains at Nilo Pecanha, this solution was viewed with disfavor. Further thickening of the lining was out of the question since the fabrication of the steel was then too far advanced for such a solution to be very attractive.

The entire problem was intensively reviewed both by the engineers directly connected with the work and by independent consultants. It was finally decided that the thickness of steel lining as revised would be satisfactory without relief drainage or stiffeners. The final design is that illustrated in Figure 2.

Comparisons and General Considerations

The foregoing presentation describes what was done in the case of two pressure shafts designed by the same organization. It seems desirable at this point to compare those shafts with others designed elsewhere. At the same time, brief mention should be made of certain general considerations, not directly connected with the design of steel linings but with which any engineer designing a pressure shaft is sure to become involved, even though full discussion of these matters is beyond the scope of this paper.

A number of pressure shafts, including those already discussed, are listed in Table 1, together with their principal features. This table is not intended to be complete but rather to show the similarities and differences between shafts constructed in different parts of the world and to indicate what might be considered common practice and to emphasize those factors that are important to design. The data shown were obtained from published reports and private memoranda. They pertain only to pressure shafts proper, and not to distribution manifolds, since the latter are generally designed with little or no reliance placed upon the rock to carry any of the internal pressure.

Geologic Considerations All of the shafts in which dependence is placed upon the rock to carry a portion of the internal pressure have been constructed in sound, durable rock of good quality. Many have been built in granites and granitic gneisses but their construction is not limited to these formations. Since the final determination of whether a pressure shaft is feasible and economical depends upon the geologic conditions at the specific site and the relative cost of alternative surface construction, it is impossible to say that any

general classification of rock is or is not suited to underground construction. The best generalization that can be made is that, if the rock can be pierced economically by usual hard-rock tunnelling methods, underground construction should be given consideration. If the rock at the site is soft or badly faulted or crushed to such an extent that full support may be required for a considerable portion of the length of the shaft or if appreciable trouble is anticipated from groundwater, then underground construction should not be considered except for security reasons or in those instances where difficult access or rough terrain make surface construction excessively costly.

Shaft Profile Shaft profiles vary within wide limits, as indicated in the fourth column of Table 1 which shows slope angles ranging from 16.7° to 90° with the horizontal. The route of the shaft is dictated in large degree by economic considerations, depending first upon the most economical location of the power station with respect to the intake and tailrace and, second, upon the cost of excavation, spoil removal and lining. Aside from the overall economics which will vary with each project and which must be determined separately in every case, the shaft must take the shortest route from intake to power station and yet must be so located as to take full advantage of the load carrying ability of the rock at depth. This requires that optimum depth be reached as quickly as possible to assure that the greater part of the shaft lies within good fresh rock of suitable quality and sufficiently removed from the ground surface to assure adequate cover. At the same time, the depth should not be excessive since greater depth usually entails greater groundwater pressure to be resisted by the lining when the shaft is empty. The natural slope of the ground overlying the route of the shaft will have considerable bearing upon the slope angle, the latter being somewhat greater, usually, than the average angle of slope of the ground.

Vertical shafts have been used rather extensively, particularly in Sweden, where economical shaft sinking methods have been developed. Inclined shafts are generally favored for high head developments. They can be constructed economically by usual tunnelling methods as described previously in the discussion of the Nilo Pecanha shaft. For these shafts, slope angles up to 48° have been used but the usual range is between 35° and 45°. Present Norwegian[2] practice lies within this range.

Considerable argument has arisen in connection with the slope angle and some authorities consider that it should not exceed about 37°. The ruling considerations appear to be safety of personnel and the free flow of spoil from the heading to the bottom of the shaft. Excessively steep slopes are hazardous to workmen both at the heading and below, due to the difficulty of maintaining one's balance on such slopes and to the danger from falling rock and tools. These difficulties can be alleviated by providing movable level-floor platforms or jumbos, which would probably be used in any event, at the heading, by providing stepped walkways with handrails in the invert and by the use of bulkheads or nets across the shaft below the heading. Flat slopes are easier to work on and of greater safety but do not allow the free flow of spoil from the heading to the bottom of the shaft.

In laying out the Nilo Pecanha shaft, consideration was given to the various arguments with regard to slope angle. To solve the problem, experimental raises were made in the power cavern area before the sloped portion of the pressure shaft was reached. It was concluded from these tests that an angle of about 42° would be satisfactory from the standpoints of safety and free flow of spoil.

Head Range The range of heads for which pressure shafts have been used is considerable, as shown in Table 1. The Fiskumfoss[2] plant in Norway, under a head of 66 feet is not listed as its shaft has no steel lining. This plant, which has been in operation for many years, represents the low-head end of the scale. The Kemano[3] plant in British Columbia, under a static head of 2585 feet is the highest head pressure shaft so far constructed. Shafts of considerably greater head are physically possible and are being actively considered.

Rock Cover The amount of rock cover provided over the shafts varies with different projects. The fifth column of Table 1 shows the approximate minimum cover provided at a number of locations. The cover ratio given is the minimum value for the vertical distance from ground surface to shaft axis divided by the internal pressure head at the same point. The values range from 0.14 to 0.5 and compare with the value of 0.7 [4]frequently used for pressure tunnels lined with plain concrete.

In the case of water tunnels with plain concrete linings, it is generally conceded that the overlying rock should be sufficiently thick to resist uplift under the full water pressure in the tunnel. Thus the submerged weight of a vertical prism of rock, equal in width to the bore diameter, should be at least equal to the total water pressure acting on the base of the prism. If the overlying rock has a unit weight of 155 pounds per cubic foot, the height of the rock prism in feet should be equal to 0.7 of the internal pressure head in feet. In cases where the rock cover is less, it is customary to reinforce the lining to take the full bursting pressure of the water.

In a steel lined pressure shaft, it would seem that partial reinforcement had been achieved and that the cover requirement could be reduced. How far one would care to go in this direction would depend entirely upon the geologic conditions at the site.

Shaft Diameter The inside diameters of shafts already constructed vary from 5'-3" to slightly over 20 feet. The size depends, of course, upon the cross-sectional area required to produce the necessary flow of water at an economic velocity. The minimum feasible size is governed by the economics of tunnelling. A finished inside diameter of 5 feet is probably as low as anyone would care to go. In North American practice a somewhat larger diameter would probably be the economic minimum. As far as maximum diameter is concerned, the limit will probably be that imposed by the water supply available for the plant rather than construction limitations.

Bore Diameter The diameter of the bore exceeds the diameter of the lining by at least the amount necessary to allow installation of the steel and to provide sufficient working space for making up the joints between sections and for concreting between the steel and rock. Additional space may be required in zones of bad ground to provide sufficient concrete for support of the rock. The diametral differences for those pressure shafts listed in Table 1 vary from slightly over 6 inches at Innertkirchen[5] to 36 inches at Kemano. The former seems excessively small while the latter is ample.

Plate Thickness Thicknesses of steel plate used for the linings do not vary over as wide a range as one might expect, considering the large differences in head and internal diameter. The thicknesses adopted at different locations are not directly comparable due to differences in opinion as to proper design, as well as to differences in the quality of rock. These differences are even more striking if the linings are compared on the basis of nominal steel stress or that hypothetical stress which would prevail if the steel received no support whatever from the rock and concrete. As shown in Table 1, nominal stresses

range from 9,800 psi to 83,000 psi. The reasons for these differences will become apparent upon considerations of the problems that arise in designing the steel to resist both internal and external pressure.

Drainage The difference in opinion with respect of drainage is considerable. This will be discussed in detail in connection with design of the steel to resist external pressure.

Design of Steel for Internal Pressure

Deformation Relationships The design of the steel lining to resist internal pressure is based upon the same concept used in the design of reinforced concrete or any other composite structure. If two or more materials are to work together in resisting load, their deformations must be compatible and the loads carried must not exceed the strengths of the materials involved. If all three materials composing a pressure shaft are in intimate contact at all times, any increase in internal pressure will result in a corresponding increase in the radial distance from the center of the shaft to a given point in any one of the three materials. Conversely, if the internal pressure is reduced, the radial distance to such a point will be decreased, provided all three materials are elastic. The amount of increase or decrease in radial distance will depend upon the load and the elastic properties of the materials.

The foregoing statement represents an ideal condition which will not be achieved in practice. For example, if the temperature of the steel lining should be reduced after the concrete has set, the steel would shrink away from the concrete, leaving a small space between the two materials. Such cooling and shrinkage no doubt occur in all shaft linings, considering the fact that the steel absorbs heat from the setting concrete and is later cooled by the introduction of water to the shaft. Also, neither concrete nor rock are truly elastic and both tend to yield under sustained load, thus their deformations under constant load will tend to increase with time. The thermal and plastic effects result in increasing the deformation of the steel relative to the elastic deformation of the rock and concrete. The deformation relationship can be expressed as follows, with reference to Figure 3 and the nomenclature of Table 2:

$$\Delta_s = \Delta_c + \Delta_r (1+k) + y_o = \frac{p_s R_s^2}{E_s T_s} \qquad (1)$$

In the foregoing equation, the value of p_s is that portion of the internal pressure which is carried by the steel alone. The value of R_s should be the mean radius of the steel lining for a rigorous treatment; however, since the linings used for pressure shafts are relatively thin, and since the entire problem of their design involves considerable estimating of values, it will be satisfactory in most cases and much simpler, to use the inside radius. The values of k and y_o represent plastic and thermal effects, respectively.

Deformation of Concrete The deformation of the concrete envelop can be determined by assuming that it deforms elastically and treating it as a thick-walled cylinder which is incapable of taking tension. The latter assumption is justified in nearly all cases as the concrete is usually not reinforced. The pressure and hence the radial stress acting on the inside surface of the concrete is equal to the internal pressure, p, in the shaft, less that portion of the

internal pressure which is carried by the steel, if the thickness of the steel lining is neglected. The radial stress in the concrete at any point will vary inversely as the radial distance and the deformation can be stated as follows:

$$\Delta_c = \frac{(p-p_s)R_s}{E_c} \log_e \frac{R_r}{R_s} \qquad (2)$$

Deformation of the Rock The deformation of the rock is more difficult to obtain but can be found by either theoretical or empirical methods. The theoretical method is based on the elastic theory, with certain reservations in recognition of the fact that a rock mass does not behave as a homogeneous, isotropic and elastic body should. This method will be dealt with first.

Theory of Rock Deformation The rock is considered to be a thick-walled cylinder, having an internal radius equal to that of the bore and an external radius equal to the shortest distance from the center of the bore to the rock surface, as indicated in Figure 3. The deformation of the bore is then determined to fit an assumption as to the ability of the rock to resist tensile stress. Two extreme solutions are possible, depending upon whether the rock is assumed to take no tensile stress or whether it is assumed that it can take all of the tensile stress resulting from the internal pressure. Under the first assumption, the deformation is calculated in the manner just described for finding the deformation of the concrete envelope and can be expressed by the same equation by substituting the ratio $\frac{R_o}{R_r}$ for $\frac{R_r}{R_s}$ and replacing E_c with E_r. If the rock is assumed capable of taking tensile stress, Lame's equation or some modification of it is generally used:

$$\Delta_r = \frac{(p-p_s)R_s}{E_r}\left[\frac{R_o^2 + R_r^2}{R_o^2 - R_r^2} + \nu\right] \qquad (3)$$

If the external radius of the rock cylinder, R_o, is large in comparison with the internal radius, R_r, which is usually the case, equation (3) reduces to:

$$\Delta_r = \frac{(p-p_s)R_s}{E_r}(1+\nu) \qquad (4)$$

Equations (2) and (3) give such widely different results, as shown in Figure 4, that it is usual to consider two concentric rock cylinders, using equation (2) for the solution of the inner one and equation (3) for the solution of the outer one with R_e (Fig. 3) as the common radius. The sum of the two deformations equals the total deformation in the rock. When this method is used, the effect of the concrete envelope is frequently ignored and the rock assumed to be in direct contact with the steel. This simplification is justified in most cases as the elastic modulus of the concrete is of the same order of magnitude as the effective modulus of elasticity of the rock.

To determine how much of the rock should be considered incapable of taking tension and hence susceptible to solution by equation (2), it is necessary to estimate the extent to which the rock is affected by the construction of the bore. This subject has been covered in considerable detail in a paper by Mr. C. P. Dunn[6]. In a discussion to Mr. Dunn's paper, Mr. B. F. Jakobsen

suggested that the outer radius of the zone in which there should be no tensile stress should be taken at three times the radius of the bore. This is in agreement with Dr. Karl Terzaghi's[7] estimate that, in intact rock, the effect of tunnel driving upon the natural condition of stress in the rock does not extend more than about one tunnel diameter beyond the bore. If this assumption is adopted, the variable term in equation (2) will have a value of 1.1 (Fig. 4).

The term in brackets in equation (3) reaches a relatively constant value of approximately 1.3 for any cylinder in which the outer radius is more than about 5 times the inner radius, assuming Poisson's ratio equal to 0.2 (see Fig. 4). Combining this value with the value of 1.1 given in the preceeding paragraph and assuming that Poisson's ratio might be as high as 0.3, leads to the following approximate equation, which is applicable to any case in which R_o is equal to or greater than $15R_r$:

$$\Delta_r = 2.5 \frac{(p - p_s) R_s}{E_r} \qquad (5)$$

It would seem that the approximate solution by means of equation (5) would simplify the calculations considerably and would be just as satisfactory as any other equation based upon theoretical considerations, since, in most cases requiring detailed analysis, the value of R_o will be at least as great as $15R_r$.

Applications of the elastic theory, similar to the foregoing have been published previously. Mr. Dunn's paper, noted above, is one example. Mention should be made also of the work of Trub[8], Kastner[9] and Hutter and Sulser[10]. Trub's paper is based upon the assumption that the rock is not able to carry tensile stresses and its deformation is therefore governed by equation (2). He develops an equation for calculating the stress in the steel. Kastner considers both equations (2) and (4) as well as the influence upon stress of the slope and depth of the overlying rock and the slope of the shaft. He also takes into account any initial lack of contact between the materials. His final equation shows how to proportion the total internal pressure between the steel lining and the rock. The paper of Hutter and Sulser considers the same factors discussed in the foregoing analysis and develops a general equation covering all conditions between those of equation (2) and equation (4) and which requires selection of the proper value for an exponent to reflect the relative ability of the rock to withstand tensile stress.

<u>Tests of Rock Deformation</u> The reliability of the theoretical approach is limited by the accuracy with which one is able to estimate the elastic constants of the rock, the limits within which to apply equations (2) to (4) inclusive and the values of k and y_o in equation (1). The modulus of elasticity of the rock measured in place by field test is usually appreciably less than the modulus determined by simple compression tests of rock cores. Because of this difference, the "effective" modulus of elasticity was referred to in discussing the design of the Nilo Pecanha and Cubatao pressure shafts. On this same basis the value of E_r in the foregoing equations is also the "effective" modulus of elasticity. It may be determined by field test or estimated on the basis of results of field tests in similar material at other locations. The value of Poisson's ratio determined by laboratory tests varies from 0.1 to 0.3 approximately. The author knows of no practical way of determining this value in the field. However, its effect is comparatively small. The plastic effect, indicated by k, can be determined by sustained and repetitive loading tests of

the rock in place. Estimates of the plastic deformation, based upon field loading tests made in the past, vary between 10% and 90% of the total deformation, depending upon the type and hardness of the rock in question and the magnitude of the load. The value of y_o depends primarily upon shrinkage of the steel due to a drop in temperature and can be estimated without difficulty. Any initial stress in the steel will also influence the value of y_o and such initial stress can be estimated readily.

Field tests are by far the most reliable means for determining the deformation characteristics of the rock. They may be made as described previously in discussing the Cubatao pressure shaft. The measured diametral deformations are plotted against the corresponding pressures to show the variation in shaft diameter with load, as shown in Figure 5. The resulting curve is then used to determine the value of Δ_r, corresponding to any value of p desired, and E_r is calculated, usually by means of equation (4) and some assumption as to the value of Poisson's ratio. Since equation (4) takes no account of the stresses due to the weight of the overlying and surrounding rock, the calculated value of E_r must be a relative rather than a true value of modulus of elasticity. In addition, the value of E_r varies with orientation as shown in Figure 5, which indicates vertical deformations about twice as large as the horizontal deformations. In the case illustrated the computed value of E_r at maximum load is either 1,800,000 psi or 3,900,000 psi, depending upon whether the vertical or horizontal deformation is used. Similar differences have been noted in test results from other locations.[11]

If it were possible to take all factors into account, the difference between the two curves of Figure 5 could probably be explained completely. Theoretical investigations of the stresses about a hold in an elastic material indicate that the circumferential compression at the sides of the bore is about 2.5 or 3 times the pressure of the overlying rock while the circumferential stress at the top and bottom is approximately zero. This in itself would lead one to expect a greater increase in the vertical diameter than in the horizontal diameter under a given radial pressure. In addition, the rock in the roof of any tunnel is probably more loosely aggregated, due to removal of the supporting material, than is the rock below the floor or at the sides and thus allows greater deformation in the vertical than in the horizontal direction when the tunnel is put under pressure.

It would seem, therefore, that while deformation of the horizontal diameter might correspond roughly to the relationship expressed by equation (4), deformation of the vertical diameter might be expressed more appropriately by equation (5), since the region in which the rock can take no tensile stress whatever is probably confined to the roof of the tunnel. Following this speculation, E_r determined by equation (5), using the vertical deformations of the Cubatao test, becomes 3,700,000 psi, which is not far different from the value of 3,900,000 psi computed by equation (4) on the basis of horizontal deformation. Other factors, of course, influence the magnitude of the deformations. For example, the properties of the rock vary to some extent with orientation in relation to the bedding or schistocity. Thus one would expect some difference in deformation between two mutually perpendicular diameters due to this inherent characteristic of the material.

The speculation of the preceding paragraph is presented for the sole purpose of assisting in the understanding of the use of field test results and not to prove any ready method for determining the true modulus of elasticity of the rock. When field test values are used, it is customary to determine E_r by

means of equation (4), using the larger of the two deformations determined by test. Once the value of E_r has been determined in this manner it may be introduced into any of the equations (2) to (4) inclusive or into both equations (2) and (3) or (2) and (4), if two concentric cylinders are assumed in order to obtain the design value of Δ_r. One might question this application. It would seem that, if equation (4) is satisfactory for finding E_r from test data, it should be equally satisfactory for applying the test data to design. To find E_r from equation (4) and then use it in a solution combining two concentric cylinders by applying equations (2) and (3) would appear to be adding a safety factor, unwittingly. Why not use the same equation for design that was used for determining E_r? All of which leads to the main question which is: why bother with the value of E_r at all if designing on the basis of a field test?

Empirical Determination of Rock Deformation For a truly empirical determination, all that is necessary are the test curves and the dimensions of the test tunnel and shaft. Since Δ_r is proportional to the product of internal pressure and radius of bore, it is only necessary to step up the value determined by test in accordance with the ratio between pR for the shaft and p'R' for the test chamber, thus:

$$\Delta_r = \Delta'_r \frac{p_r R_r}{p'_r R'_r} \qquad (6)$$

where the primes correspond to test conditions. If the field test is made by loading the rock relatively quickly to some pre-determined maximum value and then unloading, the measured deformation will not include any long-time plastic effects, in which case the value of k in equation (1) must be estimated. However, it is possible to obtain a value which includes the plastic yield by repeating the loading cycle two or three times and allowing the maximum load to remain until deformation ceases. If this latter test procedure is used the measured total deformation, adjusted to design dimensions by means of equation (6), will include the factor k.

It is possible to extend the empirical method further and make deformation tests in a chamber with a concrete lining or with a lining of concrete and steel, in which case the field loading test becomes a scale-model test of the completed structure. However, this should not be necessary since the determination of the radial deformation of the concrete and steel is not particularly complicated nor does it involve any unusual assumptions.

Summary To summarize the foregoing analysis, design of the steel to withstand internal pressure requires the equating of the radial expansion of the steel to the radial compression of the rock and concrete plus an additional amount to care for plastic and temperature effects. The deformation of the steel and concrete can be evaluated with reasonable accuracy by means of equations (1) and (2), once the allowable steel stress has been established and considering a reasonable estimate for temperature drop in the steel. The deformation of the rock can be determined by test and should include plastic yield. This deformation can be expressed in the form of equation (6) and will include the factor k. The values determined by means of equations (2) and (6) can then be inserted in equation (1) to complete the solution. If no loading test is made of the rock at the site, the values of E_r and k must be estimated on the basis of available information on similar material. Then equations (1), (2) and (5) may be used to achieve a solution.

If one wishes a general equation for simplicity of expression or for the purpose of comparing conditions at different locations, such an expression can be developed by combining the foregoing equations and using modular ratios. If the modulus of deformation of the rock, M, is taken as the ratio of $p_r R_r$ to Δ_r and if n and m represent the ratios of the modulus of elasticity of the steel to the modulus of elasticity and modulus of deformation, respectively of the concrete and rock, then:

$$(p - p_s) = \frac{\sigma_s - E_s \omega C}{2.3\, n \log \frac{R_r}{R_s} + m(1+k)} \qquad (7)$$

Equation (7) assumes that y_o is due to a decrease in temperature of the steel lining. If the lining is initially stressed, the effect of this upon y_o can be included in equation (7) by an additional term in the numerator on the right hand side. This is explained in Appendix I which gives the full development of equation (7).

Application Equation (7) gives the pressure that can be carried by the concrete and rock corresponding to a selected stress in the steel. It will be noted that this pressure is dependent upon the steel stress allowed, the probable drop in temperature of the steel, the geometry of the shaft and the deformation characteristics of the concrete and rock. It is not a function of the total internal pressure in the shaft. Once the allowable steel stress has been established and the values of $\frac{R_r}{R_s}$ and C have been determined, the rock and concrete will carry a fixed load as long as the shaft passes through rock having constant values for M and k. The amount of pressure transmitted to the concrete and rock should not exceed their safe bearing values. If the computation indicates that the pressure against the concrete and rock is too high for safety, the allowable steel stress must be reduced until the pressures reach safe values.

Having determined the pressure that can be transmitted to the rock and concrete, the difference between that amount and the total internal pressure must be taken by the steel. The required steel thickness can then be determined from equation (1).

The foregoing method of design applies only where there is sufficient rock cover to allow full reliance to be placed in the ability of the rock to carry its share of the load. In the design of distribution manifolds any support from the rock or concrete is usually ignored, except in those instances where the header is removed a considerable distance from the power cavern or the ground surface. Where sufficient rock lies between the header and the cavern, the lining for this portion may be designed on the same basis as the lining for the shaft proper. Where the header lies close to the cavern or to the rock surface, the steel should be proportioned to carry the full internal pressure in which case the design is identical to that of a conventional penstock. Reinforcement at the branches should be provided and this reinforcement can be proportioned in the same manner as for conventional penstocks.

Design of Steel for External Pressure

Buckling Resistance External pressure on the steel lining, caused by grouting during construction or by groundwater pressure acting upon the empty

shaft, causes compression in the steel. As long as the compressive stress does not exceed a certain limiting value, the lining will retain its shape and resist the load. At the limiting or critical stress, the lining will become unstable and fail by buckling in the same way as a slender column. It is necessary therefore to check the lining thickness, to be sure that it will have sufficient resistance against external pressure.

While the necessity for stability under external loading is generally recognized and all linings have been designed with the intention of safety against buckling under such loading, the author was unable to find any satisfactory published method explaining exactly how the buckling resistance should be determined. However, ample information is available on the theory of elastic stability and this provided the basis for development of a method specifically applicable to pressure shaft linings.

Buckling is always associated with bending, consequently for buckling to occur, the compressive force must be eccentrically applied at some location. In the case of a thin cylindrical shell, bending will take place if the cross-section of the shell is not perfectly circular. Since it is impossible in practice to form a steel cylinder to a true circle, it may be taken for granted that bending will occur in any shell or pipe when it is subjected to external pressure. The deviation from true circular form will increase, until some limiting or "critical" value is reached at which the shell is no longer able to retain its form and collapses. Figures 6 and 7 illustrate conditions of distortion associated with buckling.

In order to obtain an expression by which the critical pressure may be determined, the lining cross-section was assumed to become wavy in shape as shown in Figure 6. It was further assumed that the lining would be out-of-round initially as indicated in Figure 7 and that there would be a radial gap between the steel and surrounding concrete which would limit the maximum radial deflection which could occur. The radial deviations due to out-of-roundness, initial lack of contact, thermal changes and initial stress were introduced into the development and it was assumed that buckling would occur when the algebraic sum of the radial deflections, y, reached a value corresponding to a bending stress equal to the yield point of the steel. The mathematical development is based upon the work of Byran[12] and Timoshenko[13] and is given in full in Appendix II. The final equation follows:

$$\left[\frac{\sigma_y-\sigma_{cr}}{2E'}+\frac{6\sigma_{cr}}{\sigma_y-\sigma_{cr}}\left(\frac{y_o}{R}+\frac{\sigma_{cr}}{E'}\right)\right]\frac{R^2}{T^2}-\frac{R}{T}+\frac{\sigma_y-\sigma_{cr}}{24\sigma_{cr}}=0 \qquad (8)$$

where $E'=\frac{E}{(1-\nu^2)}$

Equation (8) can be solved by assuming values of critical stress, σ_{cr} and y_o and solving for $\frac{R}{t}$ when the yield point, σ_y, is known for the steel. The resulting values of critical stress can be converted to critical pressure without difficulty. This has been done for a steel with a yield point of 40,000 psi with various assumed values of $\frac{y_o}{R}$. The result is shown as a family of solid curves in Figure 9. These curves may be used directly for estimating the approximate buckling resistance of any lining. Since equation (8) gives values for critical stress, a safety factor must be applied when it is used for design.

The dotted curves of Figure 9 show similar relationships for an unconfined cylindrical shell which is forced by means of external restraints to deflect into any pre-determined number of waves, n.

If the lining is unable to resist the external pressure safely, it must be thickened or stiffened sufficiently to do so, otherwise provisions must be made to reduce the external pressure to a safe value.

Stiffening Methods A variety of methods have been tried or suggested for stiffening the lining. These may be divided into two categories; those which reduce the freedom of the lining to deflect radially by improving its circularity or by tightening the fit between the steel and concrete, and those which increase the number of waves of the deflection curve by restraining the steel at intervals around its periphery in such a manner as to restrict radial displacement or tangential rotation or both. In the first classification are: efforts to obtain a lining cross-section as truly circular as possible, grouting to fill all voids between steel and concrete, high pressure grouting to induce compressive strain in the steel to compensate for later temperature shrinkage, and pre-cooling of the lining followed by grouting. In the second classification are: anchors attached to the steel and embedded in the concrete, longitudinal stiffening ribs and ring stiffeners. These methods may be evaluated in terms of their effect upon $\frac{y_o}{R}$ in equation (8), and with reference to Figure 9.

Roundness, Thermal Effects and Pre-stress Roundness of the steel cross-section is subject to the practical limitations of shop fabrication and field erection. Care in both should result in reasonably small deviations from a true circular shape. Commercial steel pressure pipe is manufactured to a diametral tolerance of $\pm$ 1%. Large diameter shells can be produced to conform more closely to circularity and it should be possible to maintain diametral deviations within $\pm$ 1/2 inch or less in the completed lining. This would amount to about $\pm$ 0.2% for large diameters and no more than $\pm$ 1% for small diameters. Therefore that portion of $\frac{y_o}{R}$ in equation (8) which is due to out-of-roundness may vary from 0.002 inches per inch to 0.01 inches per inch, depending upon the care exercised in fabrication and erection. Cooling the steel before grouting, by reducing the temperature 90°F, would result in reducing $\frac{y_o}{R}$ by 0.0006 inches per inch. Grouting at sufficient pressure to stress the steel to 21,000 psi would reduce $\frac{y_o}{R}$ by 0.0007 inches per inch. Combining cooling with high pressure grouting would reduce $\frac{y_o}{R}$ by 0.0013 inches per inch. Therefore it is evident that extreme cooling and grouting would be less effective in increasing the stability of the lining than would measures to reduce the initial out-of-roundness. By recourse to Figure 9, it can be demonstrated that cooling and grouting to the above extent will increase stability by about 15% for any shell having a radius to thickness ratio between 60 and 175, if the shell is initially 1% out-of-round. If the shell is initially 0.5% out-of-round the increased resistance due to cooling and grouting will vary from 20% to 40%, the larger percentage corresponding to the larger radius to thickness ratio. On the other hand, decreasing the out-of-roundness from 1% to 0.5% without cooling or pressure grouting will result in an increase in stability of from 70% to 84% for the same range of radius to thickness ratio. Thus it appears more desirable to expend effort to reduce out-of-roundness than to go to extreme measures in pre-cooling and high pressure grouting. Furthermore, since nearly all of the buckling troubles so far experienced have occurred during the grouting operation, the use of high pressures should be approached with extreme caution and should not be attempted at all except with experienced personnel and adequate supervision.

Anchors Embedded anchors for forcing a pre-determined number of waves in the lining periphery appear to offer an attractive stiffening method. To force the lining to deflect in any desired number of waves, n anchors would be required. For a lining having a radius to thickness ratio of 175 and $\frac{y_o}{R}$ of 0.01, the resistance to buckling corresponds to that of a shell in which the number of waves, n, is 6 and the critical pressure is 40 feet of water. The value of n could be doubled by using 12 embedded anchors and this would increase the critical pressure to 170 feet of water. However, this type of anchor was used at Nilo Pecanha and proved to be less effective than anticipated. The anchors were a nuisance during construction. Many of them broke off and others were removed to make room for the concreting pipe. Of those that remained, a number broke off during the grouting operation and still others pulled out carrying chunks of the concrete with them and they failed to prevent buckling of the steel. The probable reason for the failure of these anchors was their rigidity which did not permit them to stretch sufficiently to follow the radial shortening of the lining when it was subjected to grouting pressure.

Stiffeners Longitudinal stiffeners of flat steel, projecting into the concrete envelope but not bonded to it, should be considerably more effective than embedded anchors. While these would interfere more with concreting, they would overcome the structural disadvantage of embedded anchors and would increase the stability of the steel considerably. Since they would allow radial deflection of the shell while restraining tangential rotation, one stiffener would be required for each half wave length. Thus the total number of stiffeners around the periphery to force a distortion of n waves would be 2n. For a lining with radius to thickness ratio of 100, at least 14 stiffeners would be required and would effect an increase of 37% in the resistance of the lining. For a lining 10 feet in diameter, the spacing of such stiffeners would be 27", which is rather close.

For linings with larger radius to thickness ratios, a greater number of stiffeners would be required and the spacing of the stiffeners would be proportionately less.

Ring stiffeners of the type commonly used on penstocks are satisfactory stiffening devices also. Their effect can be calculated by the usual methods for stiffened shells. They have similar disadvantages to the longitudinal stiffeners discussed in the preceding paragraph.

Any type of stiffening device attached to the outside of the lining increases the clearance required for placement and thus may necessitate a larger bore than would otherwise be necessary. They also increase the difficulty of any work that must be accomplished in the area between the rock and the steel in addition to making it almost impossible to fill this space completely with concrete. These considerations and the fact that a relatively close spacing of stiffeners is required to achieve an appreciable increase in stability make stiffeners much less attractive economically than they would appear to be at first glance.

Drainage Drainage to reduce the external pressure to tolerable limits, instead of thickening or stiffening the lining, has been advocated by some engineers and deprecated by others. The difference in opinion is indicated in the last column of Table 1. The reasoning behind drainage is identical to that leading to the incorporation of drainage facilities in dams. The idea would appear to be equally valid in both applications. Those who favor drains consider that groundwater, following the cracks and joints in the rock and concrete will, in time, fill whatever interstices or voids are left in the concrete

adjacent to the lining. If no egress is provided for such water, the pressure will gradually increase until it becomes as great as the original groundwater pressure at the same point before excavation of the shaft. On the other hand, if the area of contact between the steel and concrete is tapped and connected to a drain discharging into open air, the seepage water will be conducted away from the contact surface before it is able to build up any considerable pressure.

European designers are generally opposed to drain pipes or galleries paralleling the shaft[11]. Their arguments against drainage are based upon structural and practical considerations. A large drain located in the concrete envelope might result in failure of the concrete and bursting of the steel lining under the operating pressure. One such failure has been recorded[14], and has no doubt influenced thinking with respect to drains. In this particular case, the drain was a fairly large inspection gallery located immediately below the shaft and separated from the steel lining by a relatively thin layer of concrete. The concrete did not have sufficient strength to bridge the opening when the shaft was under pressure. The failure was influenced also by chemically active groundwater which apparently attacked the concrete as well as by the poor quality of the rock through which the shaft was constructed. On the practical side, drains are considered vulnerable to blocking by grout during construction and by deposition of suspended or dissolved matter during their service life. That these practical objections are valid, at least in part, is evidenced by the experience at Nilo Pecanha, previously mentioned.

Drains which should function properly and without structural risk have been installed in several shafts. The simple drainage system used at Nilo Pecanha has already been described. It is functioning satisfactorily despite construction difficulties. More elaborate systems of pipe drains have been constructed at the Wahleach[15] and Kemano developments. These are similar in detail to the drains installed in the tunnels at Fort Peck Dam[16] and which have been performing satisfactorily for many years. At the Tason development in Sweden, the steel lining is equipped at intervals with simple relief valves which open when the external pressure exceeds the internal pressure and permit the water outside of the steel lining to enter the shaft. Another interesting system of drainage is provided at the Gondo[17] development in Switzerland. In this installation, the drain is located at a point where the shaft passes through a fault in the rock, thus tapping the principal source of the groundwater.

Summary In view of the foregoing discussion, it would seem that the lining should be designed to withstand a reasonable amount of external pressure, without the aid of anchors, stiffening devices, extreme pre-cooling or high pressure grouting and that permanent drainage facilities should be provided to prevent pressures in excess of the resistance of the lining. As a criterion, the pressure used in design should be sufficient for safety under the grouting pressure required to fill the gaps and voids between the concrete and steel and this pressure should not be any greater than necessary to do the job. Stiffeners should be resorted to only at bends, where it is difficult to maintain roundness, and as emergency measures in those portions of the shaft where conditions prove to be more adverse than anticipated. Emphasis should then be placed upon obtaining a lining as nearly circular as possible, good concrete filling of the space between the rock and steel, and satisfactory grouting.

Design Details

Types of Steel The principal requirements for steel for pressure shaft linings are high ductility, weldability and, for cold climates, notch toughness. Any steel that is satisfactory for use in unfired pressure vessels will meet the first two requirements and many will meet all three. In general, the low carbon steels of ordinary tensile strength are the most suitable and the easiest to weld in the field. However, in high head shafts of large diameter, high tensile steel may be preferred in order to keep the thickness below a limit of about 1-1/4 inches to reduce welding costs, to eliminate the need for special heat treatment before and after welding and to reduce freight and handling costs when the steel must be shipped a considerable distance to the job site. Firebox quality steels are usually preferred due to the greater degree of quality control in their manufacture. The physical and chemical properties of a number of steels which have been used for pressure shaft linings are shown in Table 3. Properties of a number of high strength steels have been published elsewhere[18].

Allowable Stresses The allowable tensile stress used for design varies with different designers, but is generally higher than the stresses recommended by American codes for pressure vessels. The maximum calculated stresses in pressure shafts so far constructed vary between 50% and 80% of the yield point of the material in its as-rolled condition. The higher value was used in designing the Nilo Pecanha shaft, the reasoning in that instance being that the stress due to internal pressure, combined with any residual stresses resulting from welding and the reduction in yield point due to stress-relieving, would result in a total equal to the yield point of the plate material as it left the mill. Thus, the actual maximum total stress was allowed to reach the yield point value locally. Considering the fact that the material is highly ductile and that excessive strain is prevented by the surrounding concrete and rock, this apparently high stress should not be dangerous. In cases where the strain corresponding to the stress at or near the yield point would result in excessive compression of the rock, the allowable stress would have to be reduced to prevent this occurrence.

Welded Seams Welding is the usual method for joining the plates forming the lining. The type of joint used will depend upon whether the weld is made in the shop, the field fabricating yard or in the shaft. Figure 10 shows various types of joints. The two views at the top show the longitudinal seams at Nilo Pecanha. These seams were welded manually on the inside of the lining followed by flame gouging to sound metal and automatic welding on the outside. Circumferential seams shown at center were welded manually, both inside and outside for those seams completed in the fabrication yard, and from one side only against a back-up strip for the seams completed in place. The bottom view shows a special type of joint developed by Sulzer Brothers, manufacturers in Zurich, Switzerland. This joint is used for seams completed in the shaft and allows pressure testing for leakage, by means of tapped holes, spaced at intervals along the circumferential insert on the inside of the lining. Detailed information on welding is available elsewhere and need not be considered further here, except to call attention to published articles on the methods used for the Kemano[19] shaft lining and the tunnel lining for Palisades Dam[20].

Pre-heat and Post-heat Treatment Welding of mild steel in thicknesses of 3/4 inches or less usually presents no difficulty. On the other hand, heavier

plate and the high tensile and low alloy steels frequently require pre-heat or post-heat treatment or both to prevent cracking and hardening of the weld. Pre-heating by means of gas or oil fired burners is common practice and presents no special problems. Temperatures up to 400°F. are sufficient for ordinary carbon steels and post-heat treatment may not be necessary. Post-heating or stress-relieving generally requires more attention to heat control and, for complete refinement of the weld, considerably higher temperatures. A method of low temperature[21,22] stress-relieving, which is relatively simple to apply and is satisfactory for carbon steel plates up to 1-1/4 inches in thickness, involves heating the plate to 350°-400°F. This method was used at Nilo Pecanha and Kemano with good results. For heavier plate, large assemblies and steels which cannot be treated satisfactorily by the low temperature method, it is necessary to use higher temperatures which require more elaborate equipment. Stress-relieving of this kind should be done in the shop unless it is absolutely essential to do such work in the field. At Nilo Pecanha, the wye branches for the distribution manifold were too large to ship as completed assemblies. It was necessary, therefore, to assemble and stress-relieve them in the field. For the latter operation an oil fired stress-relieving furnace was built on the job and the pieces fully stress-relieved at a temperature of 1150°F. At Cubatao, all pieces for the distribution manifold, with plate varying from one inch to 2 inches in thickness, were shop stress-relieved at 1150°F. before shipment. A special electric induction heater was designed to allow full stress relief, at the same temperature, of all field-welded circumferential seams.

<u>Weld Inspection</u> All welded seams, whether made in the shop or the field, should be thoroughly inspected. The most satisfactory method of inspection is by radiographic means, using either standard x-ray equipment or radio-active materials. Radio-active isotopes of cobalt and irridium have proven more satisfactory for use in examination of thick plates than x-ray and have the advantage of being better adapted to field use since no fragile tubes are required. Cobalt-60 was used for all circumferential seams welded in the shaft at Nilo Pecanha. The same material was used at Kemano, where the radio-active source was held in a specially built container which allowed the entire circumferential seam to be radiographed in a single exposure.

<u>Concreting</u> Concreting of the space between the steel lining and the rock may be done by conventional methods, using chutes for inclined sections and pump placement in flat reaches, or by means of the Prepakt method. The latter has been used successfully in this type of work and it is claimed that the method reduces the amount of later grouting required. The principal thing to remember in concreting is the necessity for complete filling of the entire space between steel and rock. Reinforcement interferes with concrete placement and should not be used except where rock conditions are such that it becomes absolutely necessary.

<u>Grouting</u> Grouting after the concrete has set has been discussed to some extent in connection with the design of the steel. The first stage of grouting at low pressures should be aimed at filling any voids that might exist between the steel and concrete. This should be followed by somewhat higher pressure grouting to fill any void spaces between concrete and rock in the roof portion of the shaft. The rock in the roof section as well as throughout fissured zones should then be grouted to consolidate it and improve arch action.

In European practice grouting is considered to be one of the most essential operations in pressure shaft construction and considerable effort is expended

in attempting to obtain complete filling of all voids between the steel lining and the rock. In particular, all seepage zones are thoroughly grouted to seal off the groundwater. At least one engineer[23] advocates grouting at high pressure to induce compressive stress in the steel to offset the effects of cooling in service and of the compression resulting from groundwater pressure when the shaft is empty. It has been suggested that the grouting pressure should be equal to the ground water pressure and that grouting so performed would constitute a proof test of the lining. Such high pressure grouting must be done with extreme care and requires that the lining have sufficient resistance to buckling under the grouting pressure or that it be braced internally to resist that pressure. If braced internally, the bracing should be so designed that it will allow the required radial compression to take place in the braced areas, otherwise no pre-stress will be achieved.

Pressures used in grouting steel lined shafts in Europe are fairly high. For the space between the concrete and the steel a pressure of 85 psi was used for the Gondo shaft. For grouting the concrete to rock contact double this pressure was used. These pressures correspond to 30 psi and 50 psi, respectively, used at Nilo Pecanha.

Inspection and Supervision Before closing, a word should be said of the need for adequate inspection and supervision during the construction of a pressure shaft. No matter how well the shaft is designed its performance will be dependent in large degree upon the careful attention to construction procedures envisioned by the designer. While this is true of any large scale undertaking it is more difficult to obtain the desired results in a pressure shaft because of the multiplicity of operations going on simultaneously within a rather confined space underground. In addition, each operation takes time and there is constant pressure to maintain or improve the construction schedule. The temptation to take short-cuts is great. Strict adherence to the specifications will be achieved only by the persistent efforts of competent supervisory and inspection personnel.

ACKNOWLEDGMENTS

The design of the Nilo Pecanha and Cubatao underground plants was done by Canadian Brazilian Services, Ltd. of Toronto, Ontario. Supervision of design of both pressure shafts and the development of general principals of design was the responsibility of the author under the general direction of G. O. Vogan, Manager, engineering projects division. Adolpho Santos, Jr., AM, ASCE, hydraulic engineer for the Compania Brasileira Administradora y Servicos Tecnicos of Sao Paulo, Brazil, reviewed the design for that company and contributed to it in many ways. General responsibility for the two projects was borne by Adolph J. Ackerman, M, ASCE, Vice-President, Compania Brasileira Administradora y Servicos Tecnicos.

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APPENDIX I

Design of Steel to Resist Internal Pressure

See Table 2 for nomenclature and Figure 3 for geometry.

The deformation of steel, concrete and rock, taking into account any lack of contact between the materials, is expressed as follows:

$$\Delta_s = \Delta_c + \Delta_r(1+k) + y_o$$

The deformation of the steel alone due to internal pressure is:

$$\Delta_s = \frac{\sigma_s R_s}{E_s} \qquad (a)$$

and

$$\sigma_s = \frac{p_s R_s}{T_s}$$

therefore

$$\Delta_s = \frac{p_s R_s^2}{E_s T_s} \qquad (b)$$

and

$$\Delta_s = \Delta_c + \Delta_r(1+k) + y_o = \frac{p_s R_s^2}{E_s T_s} \qquad (1)$$

The deformation of the concrete is determined on the assumption that it is a thick-walled cylinder unable to carry circumferential tension. Therefore the stress due to internal pressure is distributed radially. The radial stress is:

$$\sigma_c = (p - p_s)\,\frac{R_s}{R}$$

and the radial strain is:

$$\delta_c = (p - p_s)\,\frac{R_s}{R E_c}$$

The total radial deformation is obtained from the stress and strain relation in the usual manner, as demonstrated below:

$$\Delta_c = \delta_c \int_{R_s}^{R_r} dR = \frac{(p - p_s) R_s}{E_c} \int_{R_s}^{R_r} \frac{dR}{R} \qquad (2)$$

$$= \frac{(p - p_s) R_s}{E_c} \log_e \frac{R_r}{R_s}$$

or

$$\Delta_c = 2.3\,\frac{(p - p_s) R_s}{E_c} \log \frac{R_r}{R_s} \qquad (c)$$

The deformation of the rock can be expressed in terms of its modulus of deformation, M, as determined by a quick loading test, and its plastic coefficient, k, which represents the proportionate increase in deformation resulting from a sustained load acting for a sufficient length of time to reach equilibrium between load and deformation. Then:

$$M = \frac{p_r R_r}{\Delta_r}$$

$$= \frac{(p - p_s) R_s}{\Delta_r}$$

and

$$\Delta_r(1+k) = \frac{(p - p_s) R_s}{M}(1+k) \qquad (d)$$

The radial deformation, y_o, is that resulting from thermal effects, shrinkage of the concrete and pre-stressing of the steel under grouting pressure. Each of these effects will be considered separately.

The radial change due to temperature variation is:

$$y_o = \omega C R_s \qquad (e)$$

where C is equal to the temperature change expected to occur in the steel after the concrete has set. For a temperature drop the sign of y_o is positive.

Shrinkage of the concrete will tend to exert pressure against the steel and will have an opposite effect to a drop in the temperature. At the same time, the concrete will tend to shrink away from the rock, leaving a small gap. The shrinkage that might be expected in any underground location is small. If it should occur to any great extent, it will be accompanied by radial cracking of the envelope which would reduce its over-all effect. Grouting after the concrete has set will certainly fill any important cracks or zones of separation. Therefore it is believed that shrinkage of the concrete envelope may be neglected.

Pre-stressing the steel by pressure grouting will result in reducing y_o as follows:

$$y_o = -\frac{\sigma_p R_s}{E_s} \qquad (f)$$

where σ_p is the compressive stress in the steel resulting from the grouting pressure and taking into account the relaxation which will occur after the grout has set.

Substituting equations (a), (b), (c), (d), (e) and (f) in the form of equation (1):

$$\Delta_s = 2.3\frac{(p-p_s)R_s}{E_c}\log\frac{R_r}{R_s} + \frac{(p-p_s)R_s}{M}(1+k) + \omega C R_s - \frac{\sigma_p R_s}{E_s} = \frac{\sigma_s R_s}{E_s}$$

or

$$2.3\,\frac{(p-p_s)}{E_c}\log\frac{R_r}{R_s}+\frac{(p-p_s)}{M}(1+k)+\omega C-\frac{\sigma_p}{E_s}=\frac{\sigma_s}{E_s}$$

Transposing

$$(p-p_s)\left[\frac{2.3}{E_c}\log\frac{R_r}{R_s}+\frac{1+k}{M}\right]+\omega C-\frac{\sigma_p}{E_s}=\frac{\sigma_s}{E_s}$$

Letting $\frac{E_s}{E_c}=n$ and $\frac{E_s}{M}=m$, then multiplying by E_s and transposing:

$$(p-p_s)\left[2.3\,n\log\frac{R_r}{R_s}+m(1+k)\right]=\sigma_s-E_s\omega C+\sigma_p$$

and

$$(p-p_s)=\frac{\sigma_s-E_s\omega C+\sigma_p}{2.3\,n\log\frac{R_r}{R_s}+m(1+k)} \qquad (g)$$

If the pressure grouting is sufficient to induce a stress approximately equal to the yield point of the steel and if this stress is relaxed about 50% after the grout has set, the effect of grouting will be approximately equal to and opposite in sign to a temperature reduction of 90 degrees, Fahrenheit. In this case the second and third terms in the numerator of equation (g) will cancel each other. If, on the other hand, the grouting pressure is low, its effect will be negligible and equation (g) will reduce to:

$$(p-p_s)=\frac{\sigma_s-E_s\omega C}{2.3\,n\log\frac{R_r}{R_s}+m(1+k)} \qquad (7)$$

APPENDIX II

Design of Steel to Resist External Pressure

See Table 2 for nomenclature and Figures 6, 7 and 8 for geometry.

Since only the steel is considered in the following analysis, the subscript s will be eliminated from the equations.

The basic equation for elastic stability of a thin shell is given by Timoshenko[13] as follows:

$$\sigma_{cr}=\frac{E}{12(1-\nu^2)}\,\frac{T^2}{R^2}\,(n^2-1) \qquad (a)$$

where $n = \frac{\pi R}{L}$ and 2L equals the wave length of the buckling curve. The problem requires the determination of n in terms of the relative freedom of the steel lining to deflect radially within the confines of the concrete envelope.

Under an external pressure equal to that which would cause failure by buckling, the mean radius of the steel will decrease by an amount y_s due to compressive stress:

$$y_s = \frac{\sigma_{cr} R}{E} (1 - \nu^2)$$

If the steel deflects into a wave form, an additional radial deflection will occur, the amount of which can be determined by comparing the mean radius of the distorted lining with that of the undistorted lining. The shape of the distorted lining can be expressed as follows:

$$y = y_m \sin \frac{\pi x}{L}$$

where y_m is the maximum radial deflection.

The length along the deflected curve for one-half wave is:

$$L_c = 2 \int_0^{\frac{L}{2}} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} \, dx$$

and the total peripheral length is:

$$S = 4n \int_0^{\frac{L}{2}} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} \, dx$$

Integrating:

$$S \approx 4n \left[\frac{L}{2} + 1.236 \frac{y_m^2}{L} \right]$$

The length S is equal to the mean circumference of the undistorted lining while 2nL equals the mean peripheral length of the distorted lining. The radial difference between the two, y_c is equal to the circumferential difference divided by 2π. Therefore:

$$y_c = \frac{4n\left[\frac{L}{2} + 1.236\ \frac{y_m^2}{L}\right] - 2nL}{2\pi}$$

$$= \frac{4 \times 1.236\ n y_m^2}{2\pi L}$$

Since $L = \frac{\pi R}{n}$

$$y_c = \frac{n^2 y_m^2}{4R}$$

The maximum radial deflection must equal the sum of the deflections due to all causes. Then:

$$y_m = y_o + y_s + y_c$$

$$= y_o + \frac{\sigma_{cr} R}{E}(1 - \nu^2) + \frac{n^2 y_m^2}{4R} \qquad \text{(b)}$$

The maximum stress in the steel is the combined stress due to direct compression and bending. The bending stress is determined by assuming that the maximum radial deflection represents the eccentricity of the direct compression. It is further assumed that failure will occur when the combined stress equals the yield point, σ_y of the steel. Then:

$$\sigma_y = \sigma_{cr}\left(1 + \frac{6 y_m}{T}\right) \qquad \text{(c)}$$

Combining equations (a), (b) and (c) and letting $E' = \frac{E}{(1 - \nu^2)}$ results in the following expression:

$$\left[\frac{\sigma_y - \sigma_{cr}}{2E'} + \frac{6\sigma_{cr}}{\sigma_y - \sigma_{cr}}\left(\frac{y_o}{R} + \frac{\sigma_{cr}}{E'}\right)\right]\frac{R^2}{T^2} - \frac{R}{T} + \frac{\sigma_y - \sigma_{cr}}{24\sigma_{cr}} = 0 \qquad (8)$$

TABLE 1 - REPRESENTATIVE PRESSURE SHAFTS

Plant	Location	Rock Type	Slope Angle Deg.	Static Head Feet	Cover Ratio	I.D. of Lining Inches	Minimum Diam. of Bore Inches	Thick. Steel Inches	Nominal Steel Stress PSI	Min. Yield Point PSI	Drains
Abjora	Norway		43	1450	0.4	90.6	107.0	0.47-0.87	32,600	31,000	No
Bjorkaasen	Norway		90	262		106.0					No
Brommat	France	Granite	90	855		(157.5 (102.4	181.5 126.4	0.47- -1.93	9,800		
Cubatao	Brazil	Gr.Gneiss	42	2357	0.4	128.0	152.0	0.52-0.91	71,500	42,000	Yes
Gerlos	Austria	Lime-stone	27.5	2030	0.2	(86.6 (63.0	106.6 83.0	0.47- -1.57	17,700		Yes
Gondo	France	Gneiss	35	1460	0.2	63.0		0.47-0.67	29,800		Yes
Guayaba	El Salvador		90	195		120.0					
Handeck I	Switzer-land		35.8	1790	0.16	(90.6 (82.7	102.6 106.7	0.39-0.47 0.95-0.98	50,500 32,600		
Handeck II	Switzer-land	Granite	31.5	1520	0.18	(88.6 (84.6	95.6 91.6	0.43-0.47 0.47	59,000	31,000	
Harspranget	Sweden	Granite	90	355							
Hjalta	Sweden			275		216.0		0.315-1.14	11,300		
Innert-kirchen	Switzer-land	Gr. Gneiss	(31.7 (7.0	2200	0.14	(102.5 (94.5	108.8 100.8	0.47 - -0.79	57,000		No

TABLE 1 (cont'd.)

Plant	Location	Rock Type	Slope Angle Deg.	Static Head Feet	Cover Ratio	I.D.of Lining Inches	Minimum Diam.of Bore Inches	Thick. Steel Inches	Nominal Steel Stress PSI	Min. Yield Point PSI	Drains
Kemano	Canada	Gr. Diorite	(48.0 (46.0	2585	0.4	132.0	168.0	0.56- -1.92	38,500	30.000	Yes
Lyse	Norway		35.0	2060		78.7	92.6- 94.6	0.39-0.71	49,500	31,000	No
Massenza I	Italy		40.4	1810		(102.0 (98.5					
Massenza II	Italy			680		94.5					
Mera	Italy			1085		(100.0 (89.0					
Montpezat	France	Granite	25.0	2075	0.26	(94.5 (90.8		0.47- -0.95	43,000		
Nilo Pecanha	Brazil	Gneiss	42.3	1100	0.5	240.0	270.0	0.56-1.125 0.69	83,000	42,000	Yes
Rossaga	Norway		45.0	800		(130.0 (110.0		0.47- -0.91	20,900	31,000	No
South Holston	U.S.A.		16.7	255	0.5	(180.0 (168.0	198.0 198.0	0.44	21,000		Yes
Tafjord	Norway		43.0	1300		69.0		0.31-0.59	33,600	31,000	No
Tason	Sweden		45.0			94.5		0.63-1.38			Yes
Wahleach	Canada		48.0	1632		75.0	99.0	0.375-1.00	26,500		Yes

TABLE 2 - NOMENCLATURE

C = temperature change, degrees Fahrenheit

E = modulus of elasticity, psi

$E' = \frac{E}{(1-\nu^2)}$

L = length of arc

M = modulus of deformation, psi

R = radius, inches

S = peripheral length of distorted lining

T = thickness, inches

c = subscript denoting concrete, or curvature

cr= subscript denoting critical condition

k = ratio of plastic deformation to elastic deformation

m = ratio between modulus of elasticity of steel and modulus of deformation of rock, also a subscript denoting maximum

n = ratio between moduli of elasticity of steel and concrete; also an integer

o = subscript denoting initial or original state or condition

p = pressure intensity, psi

r = subscript denoting rock

s = subscript denoting steel

x = any variable linear distance

y = radial deflection or radial gap, also subscript denoting yield point

Δ = radial elongation or shortening in inches

δ = strain, inches per inch

σ = stress, psi

ν = Poisson's ratio

ω = thermal coefficient of expansion, inches per inch per degree Fahrenheit

TABLE 3 - STEEL SPECIFICATIONS

Where used	Nilo(1) Pecanha	Cubatao Shaft	Cubatao Manifold	Kemano(3) Shaft	Kemano(4) Manifold	European Specifications M-I	European Specifications M-II
Specification No.	A-212B Firebox, Modified	A-299 Firebox	A-225B Firebox, Modified	A-285B Firebox	A-201A Firebox	M-I	M-II
Chemical							
Carbon,%	0.20-0.30	0.28 max.	0.20 max.	0.22 max.	0.27 max.	0.16 max.	0.28 max.
Manganese,%	0.90-1.30	0.90-1.40	1.55 max.	0.80 max.	0.80 max.	0.70 max.	0.60 max.
Phosphorous,%	0.045 max.	0.035 max.	0.040 max.	0.035 max.	0.035 max.	0.045max.	0.045 "
Sulphur,%	0.050 max.	0.040 max.	0.050 max.	0.040 max.	0.040 max.	0.040max.	0.040 "
Silicon,%	0.20 max.	0.15-0.30	0.15-0.30		0.15-0.30		
Vanadium,%			0.08-0.14				
Copper,%				0.35			
Tensile Strength	70,000-85,000	75,000-90,000	70,000 min.	50,000-60,000	55,000-65,000	50,000-62,500	58,000-71,000
Yield Point.min.							
less than 5/8"	42,000	42,000	60,000				
5/8" to 1"			55,000				
1" to $1\frac{1}{4}$"			52,000	27,000			
$1\frac{1}{4}$" to 2"	38,000	40,000	48,000		30,000		
more than 2.00"			43,000				
Elongation in 2".%	18.0(2)	21.0	20.0	30.0	29.0	24.0	22.0

(1) Steel manufactured in France
(2) 8" gage
(3) Plates 1" or less in thickness
(4) Plates greater than 1" in thickness

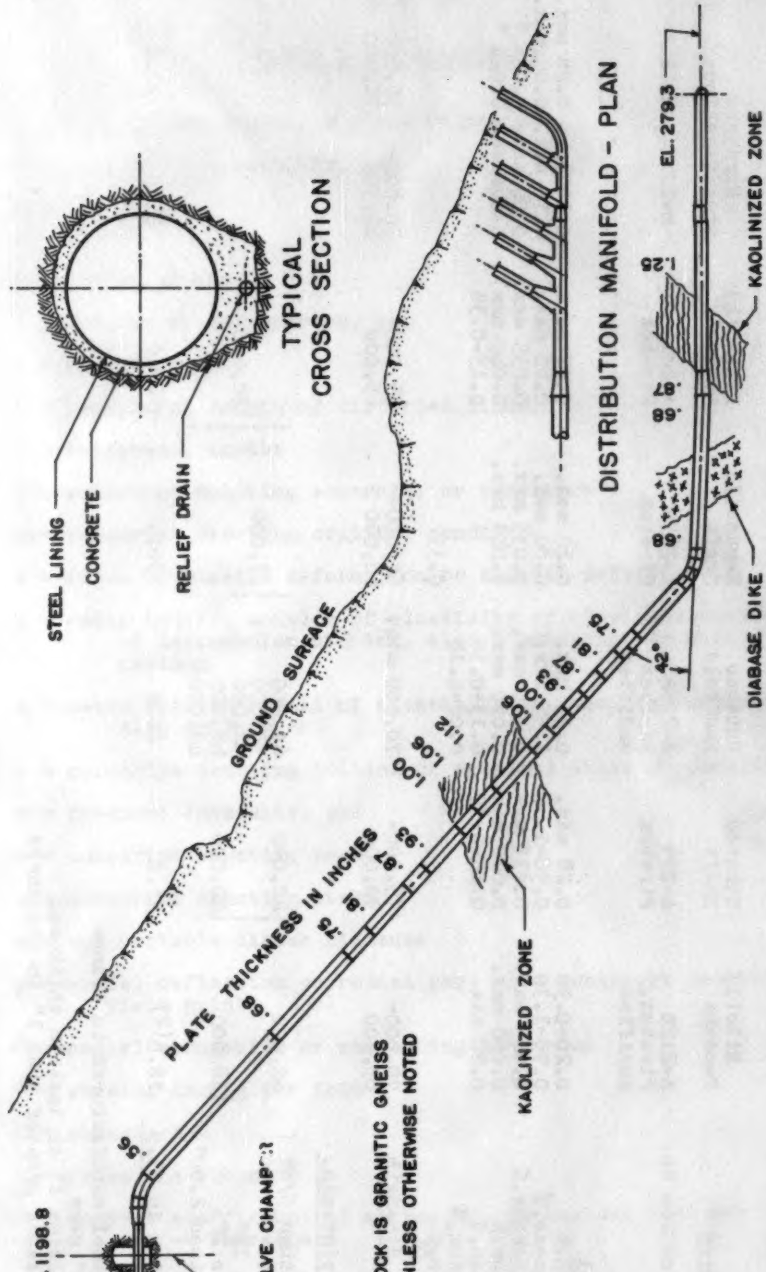


FIG. I - NILO PECANHA PRESSURE SHAFT

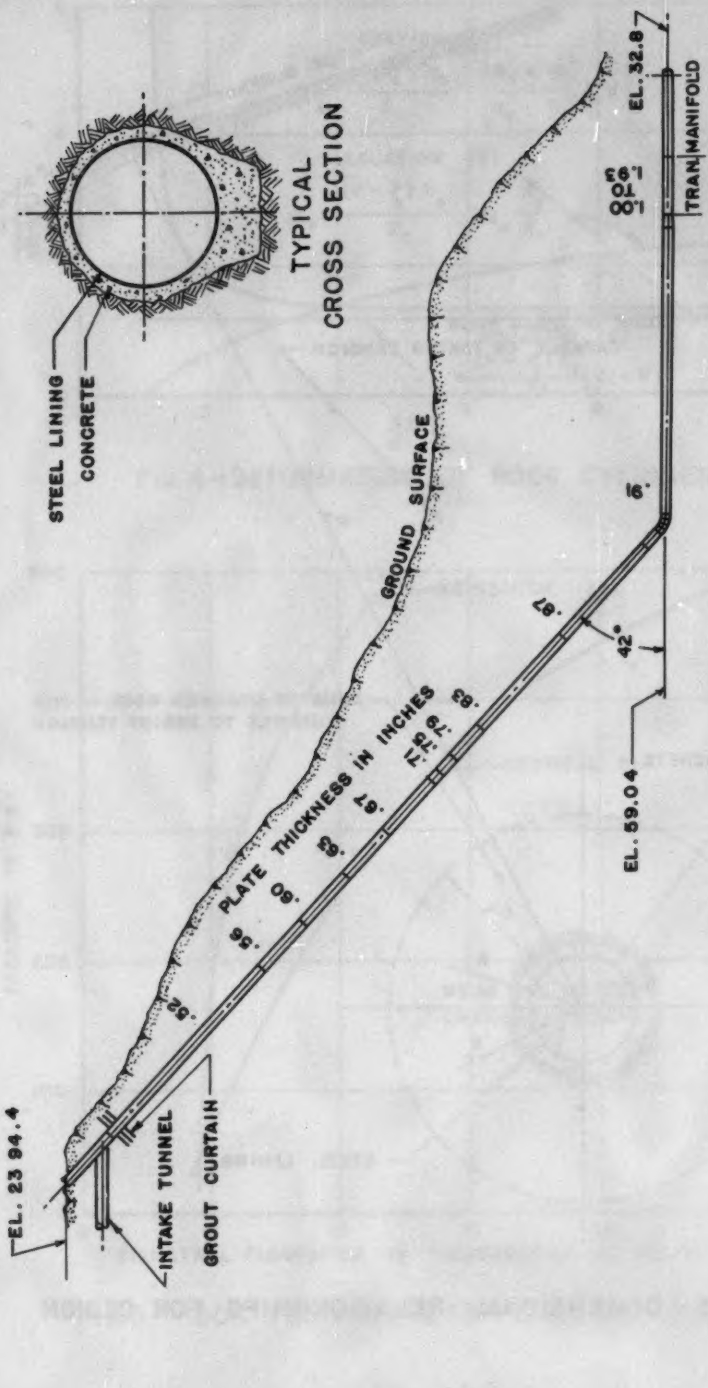


FIG. 2-CUBATAO PRESSURE SHAFT

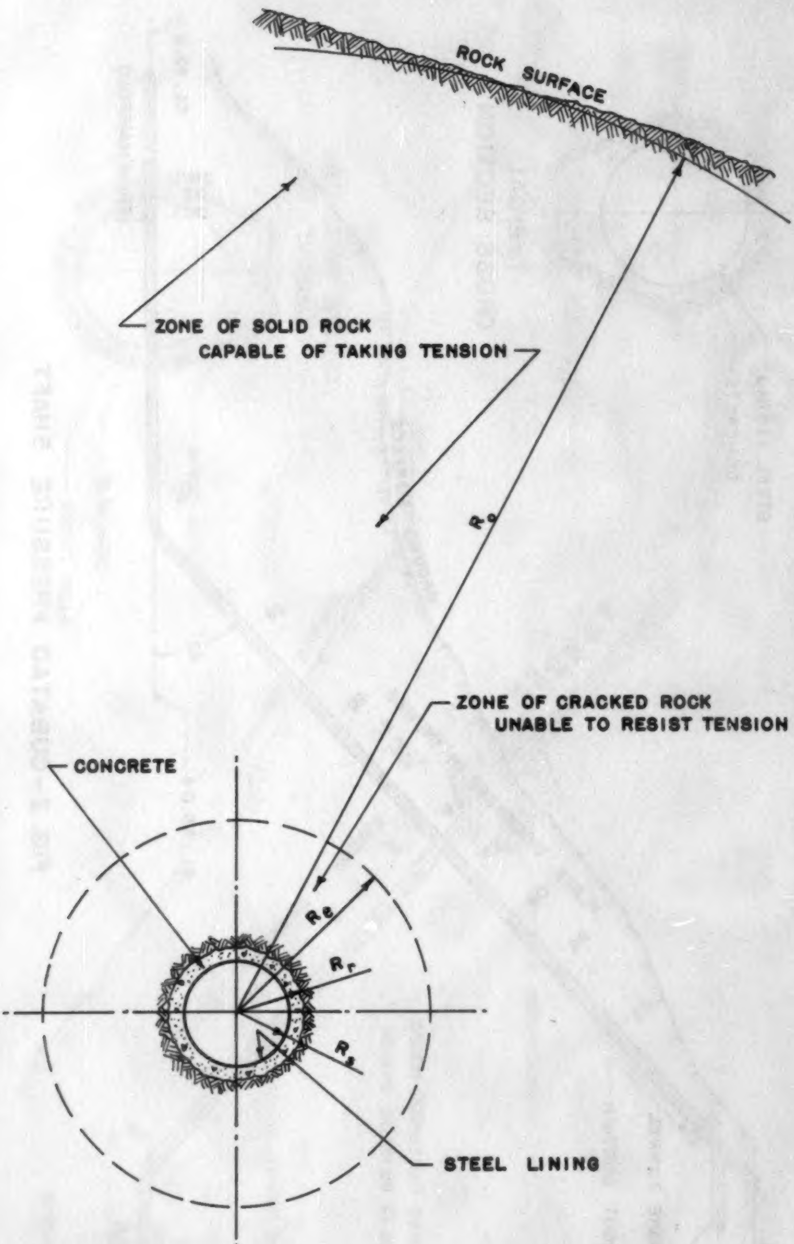


FIG.3 — DIMENSIONAL RELATIONSHIPS FOR DESIGN

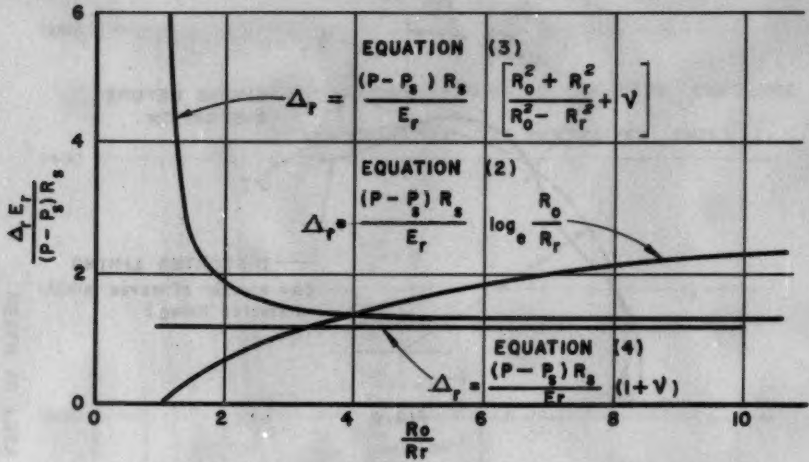


FIG. 4 - DEFORMATION OF ROCK CYLINDERS

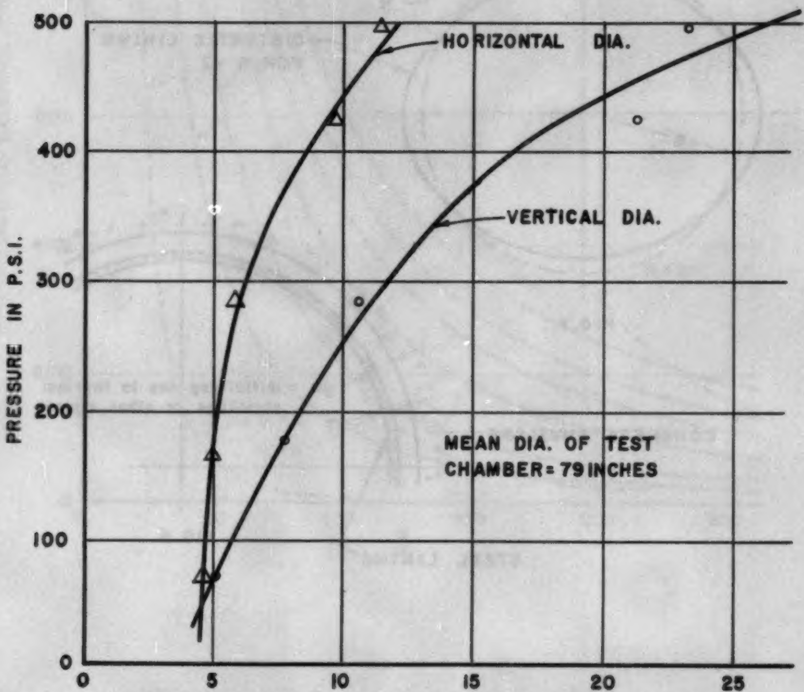


FIG. 5 - ROCK TEST RESULTS - CUBATAO SHAFT

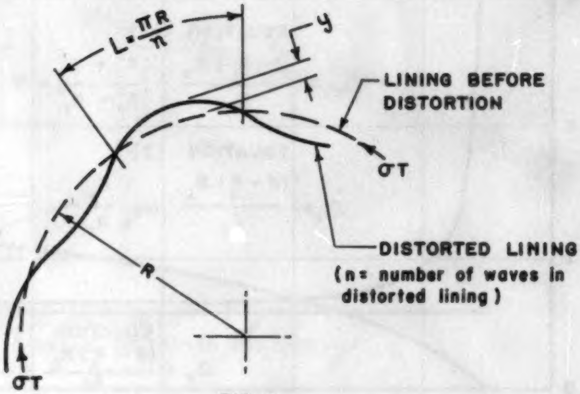


FIG. 6

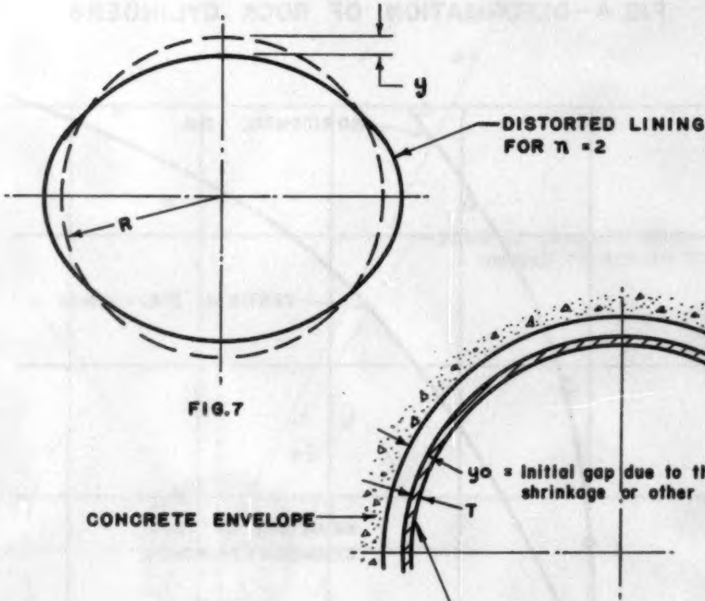


FIG. 7

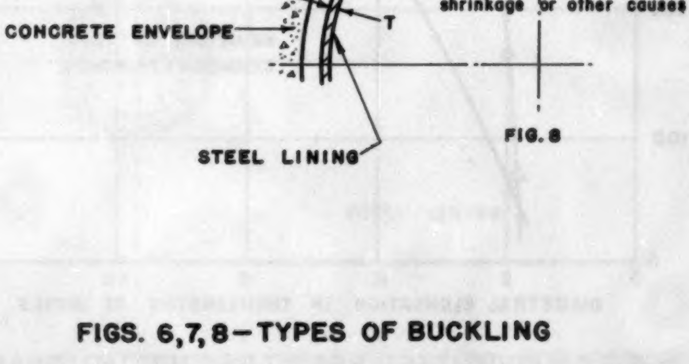


FIG. 8

FIGS. 6, 7, 8—TYPES OF BUCKLING

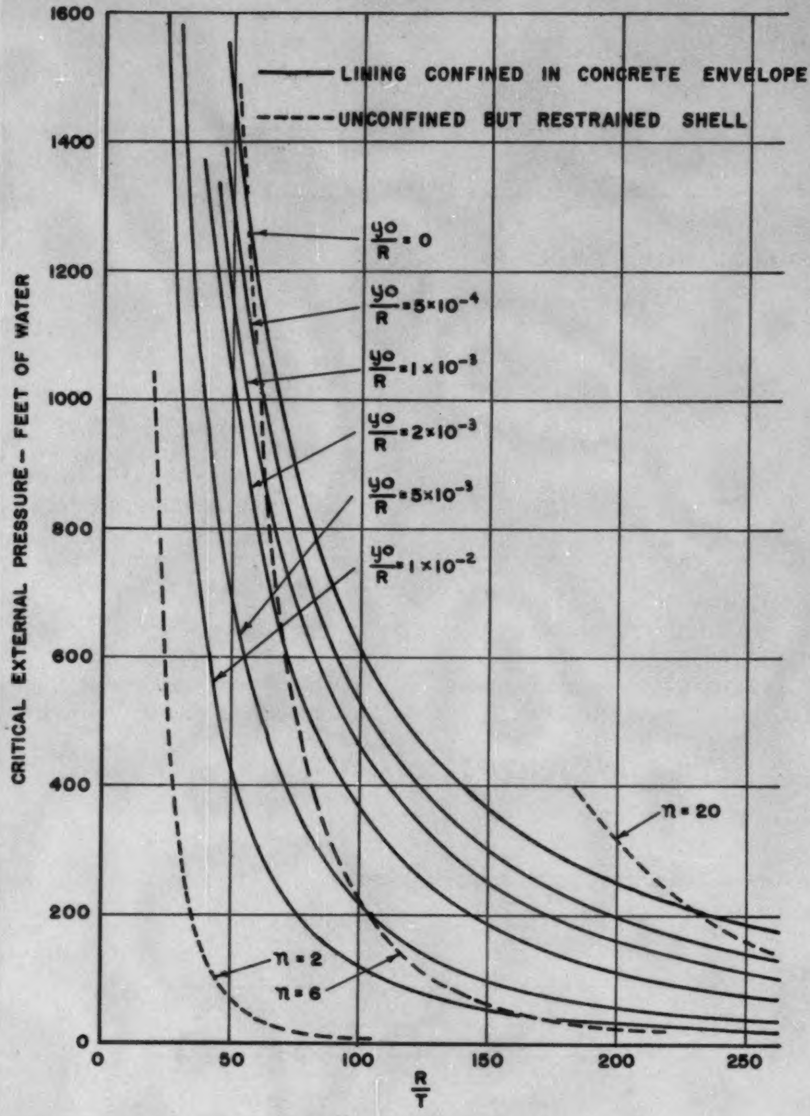
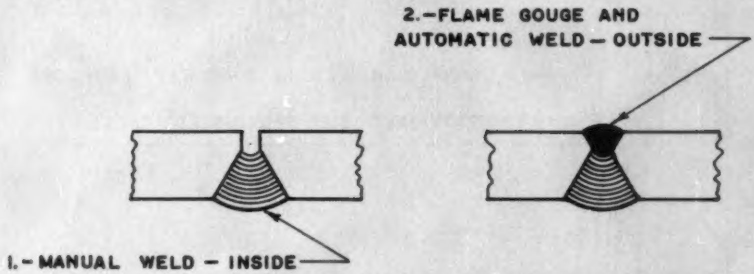


FIG. 9 — CURVES OF CRITICAL PRESSURE

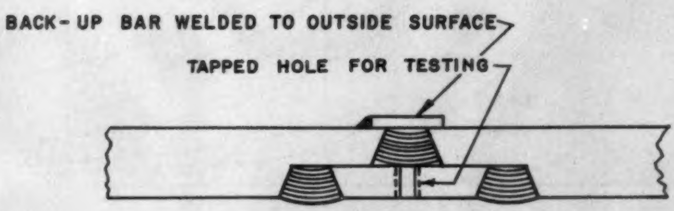


LONGITUDINAL SEAMS — NILO PECANHA

MANUALLY WELDED SEAMS MADE IN OPEN

MANUALLY WELDED SEAMS MADE IN SHAFT

CIRCUMFERENTIAL SEAMS - NILO PECANHA



SULZER BROS. PATENTED SEAMS
MADE IN SHAFT

FIG. 10 — TYPES OF WELDED SEAMS

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PROJECT CONSTRUCTION AT McNARY DAM

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(Proc. Paper 950)

SYNOPSIS

This is the fourth of a series of four papers on the McNary Lock and Dam project located on the Columbia River. The first presented the Engineering Advancements; the second, Design from Technical Considerations; and third, the Coordination of the Project Design and Construction. This paper will discuss the project construction which will include the general requirements, the general plan of construction, the construction operations, and some of the special construction features and construction problems that should be of general interest to the construction industry. The construction schedule was extremely tight, and material and labor were in short supply as a result of the Korean War. A very substantial contributing factor to the successful accomplishment of the undertaking within the scheduled time for completion was the fact that one firm was successful in being awarded all five of the principal construction contracts. Many difficult construction problems were overcome by the contractor's ingenuity and the excellent cooperation that existed between the contractor and the Corps of Engineers' field personnel.

INTRODUCTION

The main features of McNary Dam have been discussed fully in the three previous papers. For this reason, only a brief description of the project will be given in this paper. The dam is located on the Columbia River about 290 miles above its mouth. The powerhouse and spillway are in a straight line across the valley. Rock and earth embankments extended from both ends of the concrete structure to the Washington abutment on the north and the Oregon abutment on the south. The major features of the project are shown on Figure 1. The 86 x 675-ft. navigation lock with a lift of 92 feet is located on the

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Washington side. Elaborate fishladders are located on both the Washington and Oregon side. A 22-bay spillway having 50-ft. x 50-ft. vertical lift gates is located between the navigation lock and the powerhouse. The powerhouse consists of fourteen units having a total nominal capacity of 980,000 KW. The height of the structure from the lowest concrete to the powerhouse intake deck is 175 feet. Work was started on the project in May of 1947 and completed in the fall of 1954, except for the installation of generating equipment which will not be completed until December, 1956, at which time all fourteen generators will be in operation. The total cost of the project with all equipment installed will amount to $287,000,000.00.

General Requirements

The construction of the McNary Dam on a river the size of the Columbia imposed many problems in both planning and construction. Difficult construction problems arose throughout the construction period that required special attention at the time, but four major considerations affected greatly the contractor's construction schedule and operation: First, the uncertainties as to the magnitude of floods and the time at which they would occur. Secondly, existing navigation was to be interrupted to the least amount possible. Thirdly, there could be no blocking of salmon and steelhead in their upstream migration to their spawning grounds. Fourth, due to a shortage of power in the Northwest, power was to be put on the line at the earliest date possible, with December 1953 established as the target date. By excellent coordination and cooperation between the contractor and the Corps of Engineers and the willingness on the part of the contractor to meet emergencies with additional men and equipment, power went on the line as scheduled, and fish and navigation requirements were met.

Construction Phases

In order to meet the general requirements, the work was divided into three construction phases which resulted in five major construction contracts, all five of which were let to the same contractor. Figure 1 shows the general layout of the three cofferdams and the arrangement of the principal structures that go to make up the entire project. The temporary fishladders are also shown in Figure 1.

Phase One

The first contract of phase one provided for the lock excavation which was located on the Washington side. This permitted the early completion of the lock so that navigation would not be interrupted as work progressed on successive phases. A second contract in phase one provided for the construction of the first step cofferdam located on the Washington side. Most of the work under the third contract was accomplished within this cofferdam. This third and major contract in phase one provided for the construction of the lock structure, a portion of the right embankment, the Washington shore fishladder, the north 13-1/2 spillway bays, except for the top forty-one feet of the ogee section and a short non-overflow section located between the north end of the spillway and the navigation lock. Also included in the third contract was a temporary wooden fishladder through bay 1 of the spillway, and a temporary

wooden fishladder through spillway bay 13 which was attached to and parallel to the wooden rock-filled crib cofferdam extending upstream and downstream from spillway bay 13. This crib cofferdam built in the dry under the third contract became the river leg of the cofferdam required for the spillway and powerhouse construction under phase three. Figure 4 shows the first step cofferdam and the work going on within it in phase one.

Phase Two

Phase two would have been combined with the work accomplished in phase three had a shortage of power in the Northwest not existed. As a result of this shortage, it was decided to build what was called the Oregon shore junior cofferdam in advance of the main second step cofferdam, so that work could be started a year earlier on the powerhouse substructure and powerhouse intake structure of the first two units. This would make it possible to put power on the line in December 1953, which otherwise would not have been possible. The work accomplished in the second phase can be briefly described as embracing the construction of the second step cofferdam which included the main river closure, the Oregon shore junior cofferdam, and the work that was carried on within the junior cofferdam. The work within the junior cofferdam included the substructure and intake structure of the first two main generating units, and the two station service units of 3500 KW each, the substructure of the assembly bay, the temporary fishladder facilities, and a portion of the left embankment. Figure 2 shows the main second step cofferdam unwatered, the Oregon shore junior cofferdam and the work going on within it.

Phase Three

The third and final phase embraced all general construction work for the completion of the entire structure at a cost of approximately $60,000,000. The principal features included the completion of the powerhouse and permanent fish passage facilities that were only partially completed in the second phase; the construction of the powerhouse and intake structures for Units 3 to 14, inclusive; the raising of the upstream lock sill; the completion of both abutment embankments; and the raising of the concrete 41 feet in the ogee sections of the 12-1/2 spillway bays constructed on the Washington side under phase one (these bays had been left low to take the flow of the river while the second step cofferdam was in use); and the construction of spillway bays 14 to 22, inclusive. Also included in phase three was the foundation grouting program which can be divided into two parts; that under the earth structures and that under the concrete structures. Under the earth structures the objective of the grouting program was (1) to seal any cracks or joints occurring in the rock that formed the base for the silty earth core, and (2) to check the tightness of the contact of the earth core with the underlying rock. Under the concrete structures the grouting program consisted of a single line grout curtain extending from the base of the structure to the top of an impervious sedimentary inter-bed that exists in the foundation at an average depth of approximately 60 feet. The installation of all embedded turbine parts except for the last six units was also a part of the work included in phase three. A separate contract was let at a later date for placing the embedded turbine parts of the last six power units. The work under this contract is in progress at present. It is being accomplished by the McNary Dam Contractors, the same firm that did all the work under phases one, two, and three. Figure 3 shows the work under way on the powerhouse and spillway in phase three.

General Plan of Construction

A previous paper has discussed in detail the overall plan for the construction of the dam. This paper will therefore cover the subject very briefly. First, the lock was to be constructed and put in operation at a low level at the very earliest date possible by use of a temporary upstream miter gate. Secondly, 13-1/2 spillway bays were to be constructed on the Washington side in phase one, leaving bays one to twelve 41 feet low to pass the flow of the river while work was in progress in phase three within the main second step cofferdam. The north half of bay 13 was also to be left low to accommodate the construction of a temporary wooden fishladder. Thirdly, the junior cofferdam on the Oregon side was to be constructed to allow for an early start on a portion of the powerhouse. Fourth, the entire flow of the river during a low stage was to be diverted through the skeleton section of the north ten powerhouse units during the period that the ogee sections of the low spillway bays were being raised from elevation 250 to 291.

Outline of Contractor's Operation

General

The plant items and operations described below were those used on Oregon shore operations. Those in Washington were quite similar in most respects, except that the Oregon layout was generally more extensive due to greater amount of work carried on from that side of the river.

The high speed of production required, the uncertainty of river and weather conditions, the material and labor shortages resulting from the Korean War, and the difficult construction problems requiring the use of some unprecedented construction methods were all factors which made the project very difficult to plan and schedule from the contractor's standpoint. Also, the various phases of the work were so interrelated that failure to meet one of the many critical deadlines would have a "chain" reaction which could have easily delayed delivery of the first power by December, 1953 as planned, and also increased the cost by several million dollars.

The project was set up on a three-shift, six day per week basis, with a considerable amount of Sunday work necessary to meet emergencies or to complete critical work on time. As much of the work as possible was performed on day shift, or at least during daylight hours. In general, about 60 percent of the force worked on the day shift. Operations not involving precise work, such as concrete pouring, excavation, and the like were performed on the night shifts, as well as day shift, when continuous operations or conflict with the more precise work required it.

Each year the Columbia River increases in flow during the summer months with the peak in early June. The Government-designed cofferdams were designed to overtop at 690,000 c.f.s. for the first step cofferdam, 400,000 c.f.s. for the main second step cofferdam, and 850,000 c.f.s. for the Oregon shore cofferdam which was never overtopped. A flow of 980,000 c.f.s. overtopped the first step cofferdam on the Washington shore during the flood of 1948. The flow of 400,000 c.f.s. that would overtop the main step cofferdam was ordinarily reached in about mid-May and generally continued until about mid-July. However, the exact dates on which the 400,000 c.f.s. rate would be reached could vary as much as 30 days one way or the other from the average.

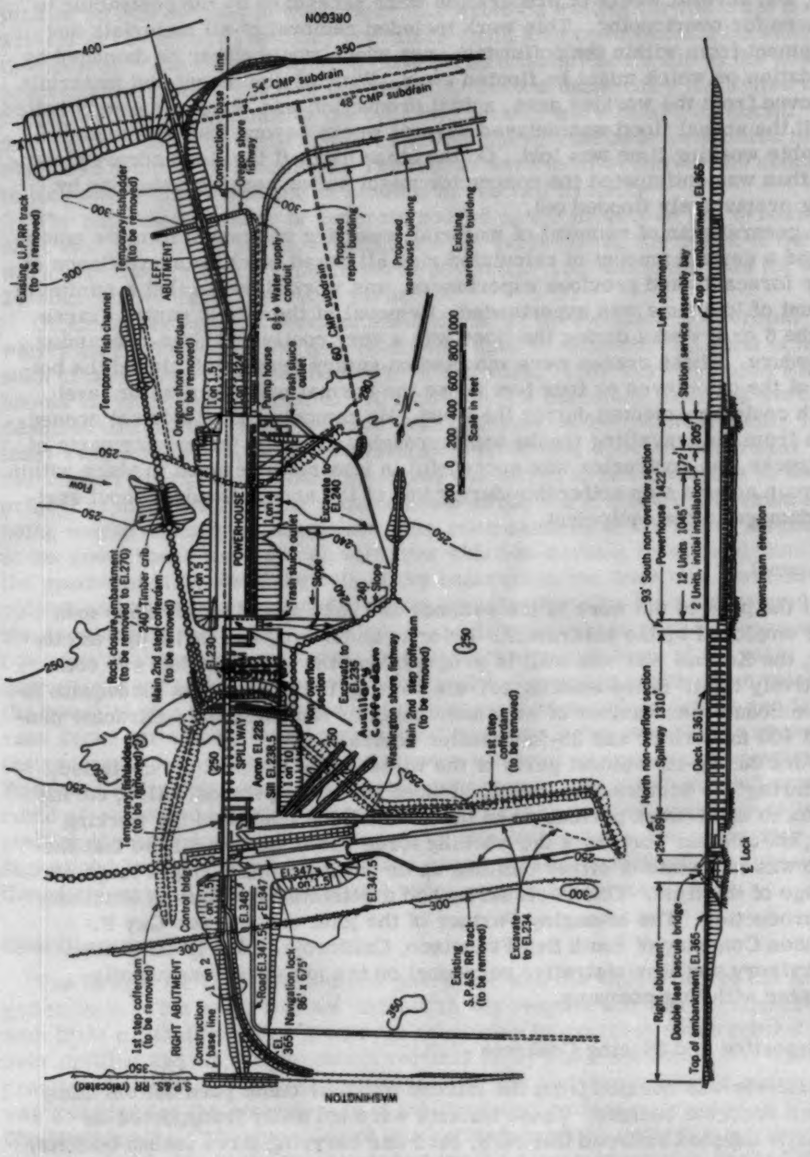

Figure 1. General layout of the three cofferdams, temporary fishladders, and the principal features of construction.

Although the effect of the storage in upstream dams, particularly Grand Coulee, was helpful in forecasting the river stages, there was ordinarily not more than two or three days' warning of flows which would overtop the cofferdam, and several weeks of preparation were necessary by the contractor to prepare for overtopping. This work included removal of all materials and equipment from within the cofferdam area which would either be damaged by inundation or which might be floated away. With all equipment and materials removed from the working area, actual production was of course nearly halted, and if the annual flood was delayed several weeks beyond that anticipated, valuable working time was lost. On the other hand, if the flood occurred earlier than was anticipated the contractor might suffer considerable loss by being prematurely flooded out.

A general plan of removal of material, readying equipment to move quickly, and a certain amount of calculated risk all based on close surveillance of river forecasts and previous experiences, was worked out so that a minimum amount of lost time was experienced. Removal of the whirly gantry cranes for the 6 or 8 weeks during the flood was a very costly and time-consuming procedure. These cranes were mounted on gantry legs which placed the bottom of the cabs three or four feet above the normal maximum water level which could be expected during the flood. By removing all electrical connections from the travelling trucks and thoroughly greasing the moving parts of the trucks, the contractor was successful in leaving the cranes in place within the main second step cofferdam during two of the annual floods without serious damage to the equipment.

Personnel

At the peak of the work in the summer of 1952 approximately 3500 men were employed by the contractor. Prior to and during the "build up" to this peak, the Korean War was still in progress and the labor market was comparatively tight. Since existing private housing in the area was inadequate to accommodate this number of workmen and their families, the contractor provided 400 modern 30 and 35-foot trailer homes.

Work during the coldest parts of the winters was necessarily curtailed, and during two winters, completely stopped for short periods. Also, during the six to eight-week periods when the annual flood inundated the working area, the greater portion of the working force had to be laid off, so that the force was in a state of either building up or cutting down during a large percentage of the time. This necessarily had a detrimental effect on efficiency and production. The Managing Partner of the joint venture was Guy F. Atkinson Company of South San Francisco, California. Nearly all of the supervisory and administrative personnel on the job were permanently affiliated with that company.

Transporting and Placing Concrete

Concrete was dumped from the mixers into four cubic yard bottom dump Garbro concrete buckets. These buckets were normally transported on specially adapted railroad flat cars; each car carrying three loaded buckets. Each flat car was pulled by General Electric 25-ton diesel electric dinky locomotive, a total of eight such locomotives being used. Each of the dinky locomotives was equipped with a two-way radio system for contact with the central dispatching point at the batch plant, enabling the dispatcher to not only

control the traffic from a safety standpoint but also to direct the locomotive operator to the proper pour location for the mix of concrete he was carrying. In addition, the operator was able to inform the dispatcher when he was returning so that batches of the proper mix would be waiting for the train on its arrival at the plant. The dinky operators could also communicate between themselves. The radios required considerable maintenance due to the adverse operating conditions, but it is felt that they were economically justifiable on a project of this size. Some of the concrete buckets were hauled on Euclid dump trucks with the dump bodies removed and special beds substituted to receive two 4-cubic yard buckets. The trucks were used to deliver the concrete to locations which would not be reached by the rail system.

The maximum quantity of concrete poured on the project on one 8-hour shift was 2,260 cubic yards, although normal rates during the height of the work averaged about 1,800 cubic yards per shift. The maximum rate for a 24-hour period was 5,604 cubic yards.

Concrete placing was accomplished primarily by electrically operated whirly gantry cranes of the type commonly used in shipyards. A total of nine such cranes were in use at the height of the work. They were equipped with booms varying from 115 feet to 140 feet in length (including jibs) and were designed to handle a loaded 4-cubic yard concrete bucket (approximately 13 tons) with a flat boom, or up to a maximum of 60 tons with the boom near vertical. These cranes were dismantled and moved to new locations on the project to meet the changing requirements of the work, some of the cranes being moved as many as six times. The main sections of the intake structure of the powerhouse were placed with four whirlies working from the ground on the upstream side, while the bulk of the concrete in the draft tube and fish collection facilities was placed by three whirlies working from the ground on the downstream side. These upstream and downstream cranes had complete coverage of the powerhouse work up to the approximate level of the main generator floor and tailrace deck. Due to the shortening radius when raising the boom, it was necessary to erect cranes on the tops of the intake and tailrace decks for setting forms and placing concrete in the powerhouse above the generator floor. The crane rails were extended as units were completed. However, after completion of the main powerhouse roof, the "outside" cranes could of course no longer service the interior work consisting mainly of installing the embedded turbine parts. For this inside work, the contractor designed and built a 78-foot span, bridge crane with 30-ton lifting capacity using the machinery out of a whirly crane which was no longer needed.

Operations in Phase One

The first work undertaken under phase one was the excavation of the navigation lock. This work was not unusual in any respect and was accomplished with little or no difficulty. It was not necessary to exercise close control over drilling and blasting to obtain vertical faces since the lock walls were gravity sections. Prior to the completion of the lock excavation, a contract was awarded for the construction of the first step cofferdam located on the Washington side. The river leg of this cofferdam which rested on solid rock near the right side of the Washington channel was constructed of steel cells, with the upstream and downstream ends flared shoreward for a distance of several cells. Upstream and downstream shore legs of this cofferdam were constructed of rock and gravel with impervious cores tied into the steel cells.

Figure 2. (Lower) First two powerhouse units under construction within Oregon shore junior cofferdam; (Center) Unwatered main second step cofferdam; (Upper) Work

This cofferdam went through one overtopping during the flood of 1948 prior to the initial construction within it without serious damage. A rock crib cofferdam, which was also a construction feature of phase one, was constructed in the dry parallel and approximately 75 feet north of the river leg of the first step cofferdam. Figure 4 shows the general layout of the crib cofferdam with respect to the river leg of the first step cofferdam. The crib cofferdam was tied into cells of the upstream and downstream legs of the first step cofferdam in such a way that by leaving these cells in when the first step cofferdam was removed, they became the means of making the crib cofferdam the river leg of the main second step cofferdam. Since the crib cofferdam tied into spillway bay 13 and since the north half of spillway bay 14 and the stilling basin below it were constructed within the first step cofferdam, it was possible to continue southward with the construction of the spillway bays and stilling basin after the main second step cofferdam had been constructed and unwatered.

The contractor's general layout for accomplishing the major portion of the work in the third contract of phase one, which embraced the navigation lock, Washington shore fishladder and the first 13-1/2 spillway bays, provided for two whirly cranes running on a wooden trestle between the two lock walls and one mounted on rail running the full length of the spillway on the upstream side. Concrete was hauled from the batching and mixing plant to the cranes on flat cars in two and four-cubic yard concrete buckets. Cranes and trucks provided the means for placing concrete in the stilling basin slab and sill and the fishladder.

Operations in Phase Two

The contract for the work to be accomplished in phase two was awarded before the completion of the work in phase one. Since the contractor was on the job he was able to start work immediately on the construction of the Oregon shore junior cofferdam. Figure 2 shows the general layout of this cofferdam and the main second step cofferdam, with construction work under way on the first two power units. The contractor's method of construction on the powerhouse and intake structure was substantially the same as that employed on the lock and spillway structure of phase one. Whirlies were placed upstream of the powerhouse intake structure and downstream of the powerhouse. Railroad cars carrying concrete had access to the whirlies upstream of the intake structure by a railroad running between the north end of powerhouse unit No. 2 and the river leg of the cofferdam.

The most difficult and interesting construction feature of phase two was the construction of the main second step cofferdam which included the main river closure in the Oregon channel and the Washington channel closure which required the construction of double rows of cells. Approximately 35 acres were enclosed within this cofferdam. Figure 2 shows the main second step cofferdam unwatered. It was in this area that the remainder of the powerhouse and spillway was constructed under phase three. Prior to the construction of the cofferdam all of the river flow passed through this area, except during high water periods when some was allowed to flow through the twelve low spillway bays constructed under phase one.

The construction of the main step cofferdam was of such magnitude that it could not be completely constructed in any single season between floods. Consequently, it was necessary to accomplish sufficient work during the

1949 - 1950 low water to permit completion during the 1950 - 1951 low water. During the first low water season, the contractor concentrated on items which were comparatively difficult and time consuming, but which were not materially restrictive to the flow of the river. These items included the eight timber cribs across the Washington channel described under the heading "Washington Channel Closure," two cells adjacent to each side of the Oregon channel in the upstream leg, one cell on the north side of the Washington channel in the upstream leg, and two cells north of the Washington channel in the downstream leg. These were completed without difficulty and suffered little damage as a result of the 1950 high water. Figure 4 shows the eight timber cribs in the Washington channel.

In the early part of September, 1950, the water had receded to a point which would permit resumption of operations. Work on the entire cofferdam was expedited with special priority given to the upstream leg in order to prepare for the final closure in the Oregon channel which was required to be complete by January 1, 1951.

The upstream cells across the Washington channel were placed by cranes working from the wooden cribs (and the bridges between the cribs) parallel to and on the downstream side of the cells. Most of the cells between the Oregon and Washington channels were placed by cranes working on the island separating the two channels. Only two cells remained to be placed between the Oregon shore "junior" cofferdam and the two cells south of the Oregon channel previously placed in 1950 in order to complete the upstream leg south of the closure opening. As many as six cranes were in use at a time in placing the cells of the upstream leg. Fill material for the thirty cells of the upstream leg north of the Oregon channel was excavated on the Oregon shore, carried across the channel by the cableway system, and dumped on the island. There it was re-loaded into trucks and dumped directly into the cells, working both north and south from about the center of the island.

The downstream leg of the cofferdam was constructed by utilizing three principal accesses. The nine cells across the Washington channel and in the deeper water immediately south of the channel were placed by a floating crane. Between the south end of that section and the Oregon channel, eleven cells of the cofferdam crossed a relatively shallow stretch of water. Access to these cells was accomplished by building an earth fill road from Artesian Island downstream (westerly) across the shallow water between the two river channels to about the center of the section of cells to be built. From there, the road was extended north and south so that these eleven cells could be constructed without the use of barges or floating cranes. Figure 2 shows the high ground between the Oregon and Washington channels as it appeared after unwatering the cofferdam. The remainder of the cells of the downstream leg between the Oregon shore and the Oregon channel were placed from an earth fill road built from the Oregon shore. The three cells across the narrow Oregon channel were built by working from the completed cofferdam on the south side of the channel. The downstream leg was completed in January of 1951, and the cofferdam unwatered the following month. The construction of the portions of the downstream leg having access by road could not be initiated until after the main closure in the Oregon channel had been accomplished.

Operations in Phase Three

The fast schedule set up for the $60,000,000 completion contract in phase

Figure 3. Work under way on powerhouse and spillway of third and final phase of construction. Note location of 8 whirlies.

three required the contractor to mobilize nine whirlies. Up until this time, the contractor was able to perform the work and keep on schedule with three whirlies. Figure 3 shows the general distribution of the whirlies and the railroads serving them both upstream of the powerhouse intake structure and downstream of the powerhouse. The upstream area was made accessible to railroad cars by constructing a trestle across the stilling basin and leaving a temporary opening through the ogee section of a spillway bay. The two whirlies on the top of the intake structure were served by the permanent railroad that crossed the Oregon shore embankment to gain access to the top of the powerhouse intake deck. A whirly mounted on rail not shown in Figure 3 placed most of the concrete in the stilling basin. The whirlies on top of the intake structure moved forward as powerhouse intake structure units were completed.

The raising of the ogee sections of the low spillway bays 41 feet from elevation 250 was one of the last major construction features of the entire project. In order to bring them up, all of the river flow had to be passing through the skeleton sections of the north ten powerhouse units at the time the flow was blocked in the last spillway bay. There was a limited time for doing the work since the work was to be initiated at a time when high water was approaching. Delays in delivery of steel for gate guides in the powerhouse created considerable anxiety as to whether the essential work on the powerhouse and intake structure could be completed in time to allow the allotted time for removing the downstream cofferdam which included the crib section below spillway bay 13. The diversion through the powerhouse and the subsequent raising of the low spillway bays could not be delayed for two principal reasons: (1) The scheduled period in which to accomplish this phase of the construction was immediately prior to high water; (2) For structural reasons, only a limited amount, approximately 160,000 c.f.s., could be diverted through the powerhouse with safety. Also a flow exceeding 160,000 c.f.s. to any great extent would overtop the upstream bulkheads used in raising the ogee sections. Although the diversion through the powerhouse was several months away, with one flood period occurring in the intervening time the tremendous advantage of an earth auxiliary cofferdam across the stilling basin from spillway bay 18 to the downstream leg of the second step cofferdam was recognized, and a decision was made to proceed immediately with its construction, the cost to be borne by both the contractor and the Government. Figures 1 and 2 show the location of this auxiliary cofferdam. With this auxiliary cofferdam in, it was possible to remove a large portion of the downstream leg of the cofferdam, including the crib section and the temporary fishladder below spillway bay 13, well in advance of the scheduled date for initiating the removal of the downstream leg. This allowed several additional weeks in which to install the gate guides and accomplish other work that had to be completed before diverting through the powerhouse. The construction of this auxiliary cofferdam assured the Government that the construction schedule would be met, and it allowed the contractor sufficient time to remove the cofferdam in an orderly manner without going to the added expense of bringing in additional equipment such as heavy floating plant. It also aided the contractor by blocking off a large quantity of water coming through the leaky crib cofferdam, thereby reducing his pumping costs over a period of several months.

The embankment was placed in the dry immediately prior to the overtopping of the main cofferdam by the high water of 1952. Both slopes were protected by rock, since both would be subject to attack by water at one time or

another. The impervious section backed up by a sand filter on the south side tied into the ogee section of spillway bay 18 and the cells of the downstream leg of the cofferdam. Immediately after unwatering the main cofferdam area subsequent to the high water of 1952, the embankment almost failed where the downstream end tied into the cofferdam cells. This was a result of the steel cell leaning slightly upstream where the core tied into the cofferdam. Evidently the core settled, and in settling parted from the overhanging wall of the cell, leaving a path through which the fine core material was washed out. It was not until after several thousand yards of a mixture of gravel, sand and silt had been hurriedly dumped into the breach that the cofferdam was saved. This near failure would not have occurred had the steel been tilting slightly downstream, thereby allowing the impervious core as it settled, to remain in contact with the steel. In the removal of this earth cofferdam, only the rock on the slopes was removed, the balance of the fill being light enough to be swept away by the water when the river was being diverted through the powerhouse.

In preparation for raising the low spillway bays, the contractor placed three of his 50-ton whirly cranes on the tracks of the two 200-ton spillway gate cranes. The three whirlies were required to meet the concrete placing schedule and also to lift the 90-ton bottom section of the spillway gates up out of their regular slots and lower them upstream of the spillway piers into the slots of the concrete pier nose extensions where they served as upstream bulkheads for unwatering the work area. Figure 11 shows the three whirlies on top the spillway. The upstream concrete pier nose extensions were extended vertically an additional five feet by the use of prefabricated steel extensions containing slots for the bulkhead. The handling of these 90-ton gates with two whirlies was a job that called for the highest degree of rigging skill. Prefabricated steel pier nose extensions that would accommodate steel stop logs were placed on the downstream slope of the spillway piers over tie-down bolts that had been embedded in the concrete. Figure 11 shows the stop logs in bays 4 and 5. The lower part of the steel pier nose extensions extended several feet below tail water, making the services of a diver necessary for placing and cinching up the nuts on the embedded tie-down bolts. Enough stop logs and steel pier nose extensions were on hand to allow for unwatering and working in five bays at the same time. When work was completed in a bay, they were removed and installed in another bay.

The downstream pier nose extensions were designed to withstand a tail water head of only twelve feet. This was adequate, provided everything went well, but work had hardly started on the first bay when the unwatering pumps failed. A head built up against the downstream stop logs from the upstream or reverse side sufficient to spread the steel pier nose extensions to the extent that all the stop logs slipped out of their slots and washed into the stilling basin.

Since a failure of the pumps could occur at any time and a freshet could overtop the upstream bulkhead which had very little free board, it was decided to stop all operations until the steel pier nose extensions could be reinforced to withstand an upstream head of about twelve feet. This delayed concrete placement for about one week which was a serious setback since the schedule allowed only 60 days to do all the preparatory work in the low bays and place the 53,700 cubic yards of concrete. There would have been a savings in both cost and time had the design of the spillway piers provided for downstream stop log slots in concrete extensions similar to the arrangement of the upstream pier nose extensions.

Figure 4. First stage construction within first step cofferdam. Note 8 timber cribs in Washington channel and crib cofferdam that became river leg of main second step cofferdam.

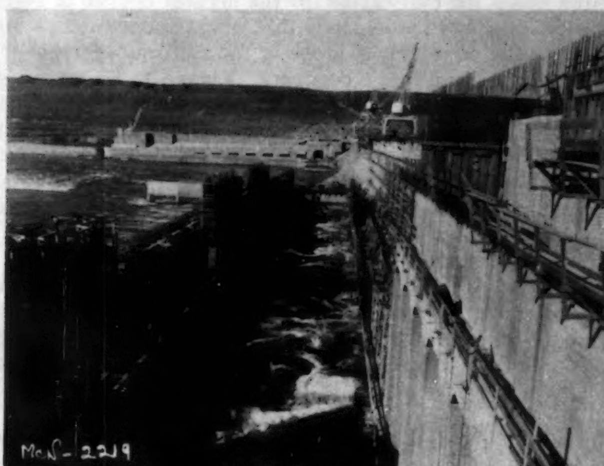

Figure 5. 146,000 c.f.s. being diverted through ten skeleton bays of powerhouse.

A temporary railroad was built across the spillway from the end of the permanent track on the powerhouse intake structure. This greatly facilitated and expedited the movement of concrete from the mixing and batching plant to the site of the work. By working three shifts, seven days a week, and increasing the lifts from five feet to ten feet, the contractor was able to finish the work on the low spillway bays within the scheduled time.

The diversion of the entire flow of the river through the ten skeleton powerhouse units was accomplished by first removing the portion of the downstream cofferdam north of the earth auxiliary cofferdam that extended across the stilling basin between spillway bay 18 and the cells of the downstream leg of the main cofferdam. Figure 10 shows the location of the auxiliary cofferdam and the portion of the main cofferdam that was removed. Next, all powerhouse intake gates were placed in their closed position, and the removal of the upstream leg of the cofferdam was initiated. The removal was accomplished by starting at the south end of the double row of cells across the Washington channel and working both ways. The upstream leg with the double row of cells at the north end and the closure fill are shown in Figure 2. The two cranes that removed the double row of cells and everything north of the initial breach were taken out by barge. Most of the sheet steel pile were hoisted to the spillway deck and hauled out by rail. The remaining portion of the upstream leg south of the double cells was removed by one crane working on top of the cells, with all cell-fill material and steel pile hauled out over the cofferdam to the Oregon shore. None of the tetrahedrons in the closure fill were removed. By the time one-third of the upstream cofferdam had been removed, work had progressed within the low portion of the powerhouse to the extent that diversion through it could be undertaken. Sufficient intake gates were removed to pass the entire flow of the river which reached a maximum of 146,000 c.f.s. Just prior to the removal of the gates a small shallow pilot channel was cut through the earth auxiliary cofferdam. This earth cofferdam was rapidly carried away as the flow through the powerhouse increased. Figure 11, a downstream view of the entire project, shows the water discharging downstream of the powerhouse.

The violent action of the water rushing through the powerhouse caused great damage to the reinforcing steel that was protruding four to ten feet from the concrete in areas where additional concrete was to be placed. Reinforcing steel costing $40,000.00 to install initially, was lost either by being broken off at the concrete or damaged by bending and twisting. The cost of replacing this reinforcing steel, including the cost of the steel, welding and drilling and grouting, amounted to $95,000.00. It is now apparent that the steel should have protruded only to the extent that was necessary for making either a butt or lap weld. The action of the water as it flows through the powerhouse is shown in Figure 5.

Upon completion of the raising of the ogee sections and the removal of the upstream leg of the cofferdam, all powerhouse intake gates were lowered and the reservoir started filling. Considerable work remained to be accomplished on the main structures, which included the completion of the powerhouse and the raising of the upstream navigation lock miter gate sill block from its temporary intermediate level to its final elevation and the installation of the permanent upstream miter gate.

River Closures Main Second Step Cofferdam

General

There were two channels in the river at the site of the dam—the "Oregon" channel approximately 60 feet in depth, and the "Washington" channel approximately 50 feet in depth. Both the Oregon and Washington channels intersected both the upstream and downstream legs of the main second step cofferdam which enclosed the work that was accomplished in the completion contract. The main and final closure of the river, which was made in the upstream leg of this cofferdam in the Oregon channel using concrete tetrahedrons, was devised by the Government which consequently was responsible for its success or failure. On the other hand, the "closure" of the Washington channel (accomplished prior to closure of the Oregon channel) was the full responsibility of the contractor. Figure 2 shows five of the eight cribs constructed by the contractor to aid in making the closure in the Washington channel and also the completed Oregon channel closure fill.

Washington Channel Closure

At the point in the Washington channel where the upstream leg was to be constructed, water velocities from five to seven feet per second in the 50 ft. deep channel made it virtually impossible, without some special expedient, to place the individual steel sheet piles making up the cells of the cofferdam. The system devised to stop the flow was to sink a series of eight wooden cribs 30 ft. by 60 ft. in plan dimensions, across the channel with 30-ft. openings between the cribs. Figure 4 shows the location of these cribs. Then, after completion of the cribs, concrete stop logs would be placed so as to horixontally span the openings left between the cribs, thereby stopping the flow of water. The main cellular cofferdam could then be constructed adjacent and parallel to the series of cribs without the interference of the swift water.

The cribs were built of a series of courses of 12" x 12" timbers, laid in a criss-cross fashion on 10-ft. centers each way. Four 60-ft. timbers were laid parallel to each other ten feet apart forming one course, the next succeeding course consisting of seven 30-ft. long 12 x 12 timbers, also ten feet apart and perpendicular to the previous course. The completed crib would then be in the form of a box 30 feet wide, 60 feet long and varying in height up to 60 feet, depending on the depth of the water where the crib was located. Each of the four vertical sides was sheeted with 2-inch lumber on the inside to contain the fill material. The four 9-ft. square "Compartments" in each corner of the crib were also sheeted to form pockets into which rock was dumped to hold the cribs level during their construction. The bottom of the crib was a built-up section of 12 x 12 timbers running both horizontally and vertically with considerable cross bracing, fashioned in a manner to fit the contour of the irregular bottom of the river. These first courses, up to the point where full length 30-ft. and 60-ft. timbers were used, were built in an upside down position on specially constructed ways so that they could be launched in the same manner as a ship. After launching, these sections were righted by a crane near the water's edge, and then towed to the approximate final location of the crib. There, lines were attached to the two upstream corners, the lines running 500 feet upstream to a winch, which was set up on a small island. The exact position of the cribs could then be controlled by adjusting these lines. The successive courses of 12 x 12 timbers forming

the body of the crib were then placed on top of the original bottom course (using 1"x 22" drift pins) in the manner described above, the pockets in the corners being built up as each course was added. As more timbers were added and the crib tended to sink itself of its own weight, the vertical alignment was controlled by placing rock in the proper pockets while the anchor lines held the crib in its proper location. When soundings indicated that the bottom of the crib was nearing the bottom of the river, 20 or 30 cubic yards of rock was dumped in the main portion of the crib in order to completely sink it. By means of tug boats and the upstream anchors, the crib was then "jiggled" until the contours of the tailor-made bottom fitted into the corresponding contours of rock. The crib was then completely filled with rock and the anchors cut loose. Even the largest cribs, 60 feet high, built and placed in this manner were stable enough to withstand the swiftest waters of an annual flood.

As each crib was completed, temporary bridges were built between the cribs consisting of a series of trussed wooden girders and capable of carrying the 76-ton weight of a 2-1/2 cubic yard shovel rigged as a crane. In this manner each crib, during construction, could be reached by the crane.

After completion of the eight cribs, specially constructed concrete stop logs were placed between the cribs. These stop logs were cylindrical in shape, 30 inches in diameter, 34 feet long and weighed 22 tons. They were moderately reinforced and contained embedded steel eyes in each end for handling. A 60-ton capacity whirly floating crane was used for placing the stop logs on the upstream side of the cribs so as to span the 30-ft. opening between the cribs. The stop logs, being round, rolled down the sides of the cribs. This was a much easier process than attempting to slide or force down, flat-sided stop logs against the friction caused by water holding such stop logs against the cribs.

No slots were provided in the cribs to receive the ends of the stop logs and difficulty was encountered when the lower stop log could not lie level due to the uneven bottom. The succeeding stop logs then had a tendency to slip endwise, hence losing their bearing against one of the cribs. Also, it was later discovered that during a subsequent period of low flow in the river there was insufficient force against the stop logs to hold them against the cribs. For these reasons it is now felt that some type of slot in the cribs themselves would have warranted the additional cost. The problem was solved by driving two "T" steel piles vertically near each end of the upstream side of the first stop log after it was placed and then tying them back to the cribs with wire rope. The total cost to the contractor for constructing and removing the eight cribs was approximately $1,000,000.

With the main flow of the water stopped, the main cellular cofferdam, adjacent to the cribs was constructed without difficulty. Individual sheet piles were pre-cut to length based on soundings, loaded on a barge and anchored at the site. Piles were strung and driven in the usual manner by cranes working from the cribs and the bridges between the cribs. Four cranes working simultaneously on different cells were used on this portion of the work due to the critical time schedule.

In construction of this and all other portions of the cellular cofferdams, the individual piles were driven with double acting air-operated pile hammers. The shape of the cells was controlled by stringing and driving the individual piles around movable templates. The templates consisted of two circular frames, one 10 feet directly above the other, both supported on four vertical

extra heavy 6-inch pipe spuds running through both frames. The bottoms of the spuds rested on the river bottom, the position of each spud with respect to the frames being adjustable to allow for the variations in the river bottom. In placing and driving, the piles were merely placed against the outside of the template. The templates also afforded access to the cells during construction.

Main Closure—Oregon Channel

Considerable has been written describing the main river closure in the three previous papers. To avoid repetition, this paper will treat only with the construction procedure and some of the problems encountered in the placing of material and effecting the closure.

It is generally agreed that the McNary closure was the most difficult river closure ever attempted. Considerable anxiety existed as to whether or not it would be successful as originally planned. Although model studies indicated that it could be accomplished with 8-ton tetrahedrons, it was finally decided to use 12-ton tetrahedrons to provide a greater factor of safety. The responsibility for making the closure rested with the Government. The contractor furnished all equipment, labor and materials and was paid a unit price for each tetrahedron, and a unit price for each ton of rock, gravel, sand and silt that went into the closure fill. The contractor's principal piece of equipment was the cableway which was comprised of the three-inch main load line and skip and the two 120-ft. high movable towers. The towers were spaced 1700 feet apart and mounted on heavy tracks permitting a maximum travel of 260 feet. The traveling tail tower was located on an island on the north side of the closure channel. The closure opening and the contractor's cableway for handling the tetrahedrons and other fill material is shown in Figure 7. By the use of a travel indicator, actuated from the four-foot sheave of the 1-1/8" diameter continuous haul cable and a graduated base line located near the sloping front track of the control tower on the Oregon shore, the operator stationed in the control booth was able to dump the skip within five or ten feet of a predetermined location. A government inspector was in the control booth at all times for the purpose of giving the operator the coordinates for dumping each skip load. The dumping schedules were worked out well in advance of the actual placing. This close control on placing was necessary since sounding the deep fast water with any degree of accuracy was practically impossible during certain stages of the construction. Velocities varied from 10 feet per second at the beginning to 25 to 30 feet per second at the critical stage. Accurate sounding would not have been sufficient for controlling the placing of the upstream filter blankets in water up to 60 feet in depth, since there had to be complete coverage with each layer of the various sizes in the filter blanket. Otherwise there could have been excessive leakage through the closure fill which became a part of the cofferdam. The construction schedule allowed for less than sixty days in which to make the closure. Had it been unsuccessful, the completion of the entire project would have been delayed one year. Every precaution possible was taken to insure its success. All the tetrahedrons had been cast prior to the starting date of October 10 and all the required rock was stockpiled close by. Heavy cable was on hand to provide for anchoring the tetrahedrons if their drift was excessive at the critical velocities. Anchoring would have involved the placing of a heavy anchor cable across the 240-ft. span between the steel cells at each end of the closure opening.

A model on a scale of 1 to 24 was constructed on the job for the purpose of assisting in determining whether the tetrahedrons and back-up B-stone were taking shape under the fast flowing water as was anticipated. The placing of the fill in the model was carried on simultaneously and precisely in the same manner as in the prototype. This was a great help for those responsible for determining the placing schedules, since the soundings that were made in the swift water by working from the skip were very unreliable. Although the behavior of the material as it was placed in the model was not entirely indicative of what was going on in the prototype, it was remarkable how close the sections resembled each other when it was possible to compare them after unwatering the cofferdam.

The general plan for the closure was to effect an initial closure that would block the flow, but still permit a considerable amount of water to pass through the voids. This was to be accomplished by placing alternate lifts of tetrahedrons and B-stone, at all times keeping the tetrahedrons downstream of the stone and at least five feet above the stone. This stone referred to as B-stone was to act as a back-up for the tetrahedrons, thereby reducing the quantity of the more expensive tetrahedrons. See Figure 8 which shows a cross section of the closure fill. By keeping the tetrahedrons higher than the rock, the rock, which had a greater tendency to move in the swift water, was restrained from being washed over the downstream slope where they would serve no useful purpose. When a long slick formed on the water surface below the crest of the fill, it was an indication that a low spot with higher than average velocities had formed. This was a signal to disregard the placing schedule temporarily and concentrate on dropping tetrahedrons in the low spot until the slick had disappeared. In the absence of slicks, the water surface had the same appearance for the entire width of the closure. Since soundings during the initial closure were unreliable as the result of the fast flowing water and the extremely irregular surface of the submerged tetrahedrons, it was decided to measure the drift of the tetrahedrons to determine whether the fill was coming up as planned. The method of observation was the following. A double-cone buoy was fastened with a 1/8" diameter airplane cable 200 feet long to the lifting eye of a tetrahedron while the skip was on the shore at the loading point. The tetrahedron was then hauled out to its dropping point with a cable and buoy trailing behind. The current swiftly carried the buoy downstream and tightened the cable. Before dropping the tetrahedron two transits cut in the location of the buoy. After dropping the tetrahedron, the new location was cut in. The drift of the tetrahedron was indicated by the downstream drift of the buoy. The drift did not exceed 80 feet on any of several dozen tetrahedrons dropped with buoys attached.

Due to the very tight closure schedule, the contractor had planned to carry out the closure in the Washington channel concurrently with the construction of the main closure fill. As a result of a sudden rise in the river, the closure of the Washington channel did not work out as well as was hoped for, which resulted in complete stoppage of construction work on the main closure fill for sixteen days after the first 439 tetrahedrons had been placed. During the 16-day stoppage the contractor was able to get caught up with the closure of the Washington channel.

When work was resumed on the main closure, no particular difficulties were encountered in bringing the tetrahedron and rock sections up to where they emerged above the surface of the water. At this time the river flow was 111,000 c.f.s. with a headwater elevation of 269.35 and a tail water elevation

Figure 6. Main Oregon channel closure showing the closure opening and the traveling head and tail towers.

Figure 7. Initial closure after dropping 2,088 12-ton tetrahedrons.

of 253.80, making a total differential of 15.55 feet. The river flows during the closure varied from 96,000 to 157,000 c.f.s. Upon completion of the initial closure, which took 2,088 tetrahedrons, the closure section was immediately brought up to elevation 276 by placing additional tetrahedrons and rock. This was done to protect against the possibility of overtopping from a sudden rise in the river. Figure 7 shows the initial closure with tetrahedrons protruding above the surface.

Prior to placing the C-stone, measurements were made to determine the flow through the fill. It was found that the leakage through the voids in the B-stone and tetrahedrons amounted to approximately 12,000 c.f.s. The sealing off of this flow of water became a major operation, in many respects more difficult with more uncertainties connected with it than the actual blocking of the closure opening with the tetrahedrons and B-stone. Figure 8 shows the successive layers of material placed upstream of the B-stone. First, "C" rock was placed which was overlain with four feet of spalls, followed with six feet of bank run gravel, then four feet of No. 4 to 3/4" gravel, a four-ft. layer of No. 4 minus sand, and then a layer of impervious material made up of gravel, sand and silt capped with three feet of dumped stone revetment. The gravel and sand used for the two four-ft. layers of filter material were produced in the contractor's concrete aggregate plant. All gravel, sand and silt were placed by lowering the skip to the bottom before dumping.

The section of the upstream blanket as it was actually built departed considerably from the original plan, in that the original plan for sealing off the flow called for placing only a 10 to 12-ft. layer of a mixture of gravel, sand and silt over the spalls. This plan was carried out initially, but since the placement of the material was started at the toe of the spalls in approximately 60 feet of water, it was not until the last few skip loads were being placed near the surface of the water, that it was realized that all of the sand and silt and some of the smaller gravel particles had been sucked through the spalls and lost. This, of course, made the blanket ineffective as a seal, but the gravel that remained did serve as a filter layer for the revised plan for sealing. Measurements taken in the channel below the closure indicated that 800 c.f.s. was percolating through the blanket of gravel, sand and silt that was to have sealed off the flow.

The four feet of No. 4 to 3/4" gravel and four feet of No. 4 minus sand filter zones were laid down in 2-ft. layers, the theory being that the chances of getting full coverage with each filter zone would be increased as the number of layers was increased for any one zone. Twelve inches each of both the sand and 3/4" gravel zones would have been sufficient, but the material was being placed under water which made complete coverage extremely difficult to accomplish. As an additional precaution to assure complete coverage with the sand and 3/4" gravel filter zones, the total under-water area of each was dragged with a heavy steel drag before placing the next zone. This was accomplished by lowering the drag to the toe of the slope with the cableway and then dragging it up the slope to the water surface with a crane located on top the closure fill.

An attempt was made to place the thick bed of impervious material made up of the mixture of gravel, sand and silt by end dumping off the top of the closure fill with dump trucks, with the hope that it would become partially saturated and in that state, slide down the slope and spread itself out over the entire area of the 4-ft. bed of filter sand. The only advantage of such a procedure was that if it worked, the cost of placing the impervious blanket

would be considerably less than lowering it to the bottom by use of the cableway and skip. The scheme was wishful thinking. It was not only unsuccessful, but caused considerable damage to the 4-ft. sand filter bed. The outside slope stood up steeper than the slope of the filter. Small portions broke off from time to time, but not in the manner hoped for. All of a sudden the entire mass moved down the slope and beyond the toe. A diver was sent down to determine what had taken place. It was discovered that most of the material had moved out and beyond the toe of the slope and that the slide had stripped off all the filter sand from a large area of the slope. The sand filter was restored and the impervious layer deposited in place in the same manner as followed in placing the filter beds, which required the lowering of the material to the bottom before dumping.

The amount of percolation through the closure fill proved to be remarkably small, considering that it was 240 feet long with a head against it of 60 to 70 feet when the cofferdam was unwatered. The measured flow amounted to approximately 30 c.f.s.

In making the closure, it was demonstrated that with river conditions such as those that existed at McNary, as much or more effort should go into planning the method of making the embankment impervious as in planning for the blocking of the flow in the initial closure.

Installation of Powerhouse Equipment

The installation work at McNary Dam has been carried on by a great many organizations, represented by the following: McNary Dam Contractors, Cascade Constructors, General Electric Company, Guy F. Atkinson Company, Premier Gear & Machine Works, numerous supply contractors and Government forces. In general, the installation of the gates, trashracks and stop logs was included in the construction contract.

The turbines and governors, the main control system, the 15 kv bus and air circuit breakers, the connections to the main transformers, and the 230 kv bus, as well as other main unit auxiliary equipment are being installed by Cascade Constructors. All equipment installed by the Cascade Constructors is government furnished. The main unit generators are being supplied in place by the General Electric Company and the English Electric Export and Trading Company, Ltd., with the General Electric Company furnishing 12 of the 14 units. The field assembly and placing of the main transformers is being accomplished by Government forces.

The erection of the main units is of particular interest, especially from the standpoint of the precision with which the work must be accomplished. The embedded parts have been set to the close dimensions indicated in the following table:

	Initial Check Out Before Concrete Pour	Average Final Check After Concrete Pour	Max. Final Check After Concrete Pour
Discharge Ring - Roundness	.006"-.007"	.009"	.015"
Stay Ring - Level	.004"-.005"	.012"	.026"
Stay Ring - Roundness	.008"-.009"	.025"	.030"

Discharge Ring diameter is 280 inches.
Roundness dimensions are maximum difference between radii.
Level dimensions are sum of the plus or minus deviation from a level plane.

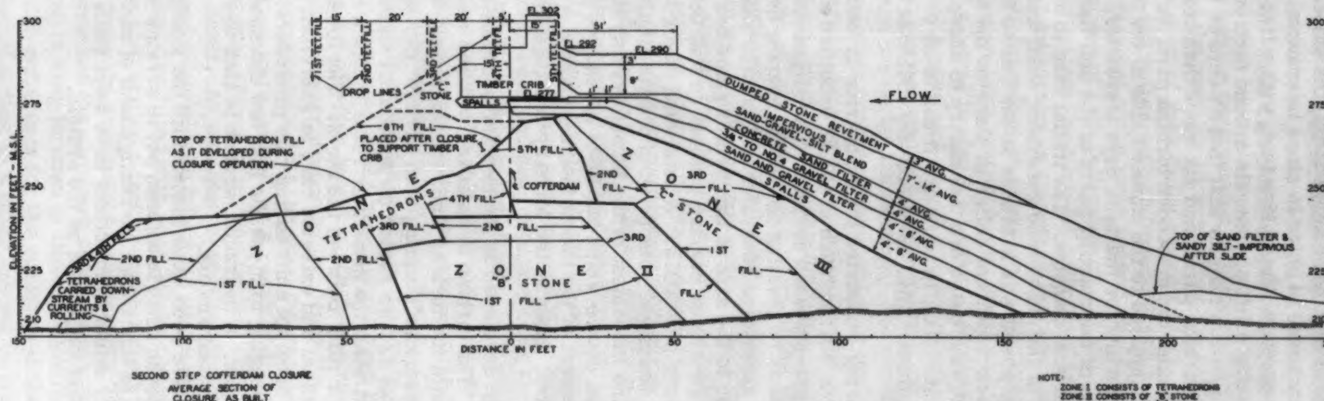

Figure 8. Cross section of the main Oregon channel closure fill showing timber crib. Note layers of filter zones.

To obtain the accuracy set forth in the above table, the embedded turbine parts were first securely anchored in place to close tolerances. Concrete was then placed around the parts in lifts of five feet, with a five-day cooling period between lifts. In placing a lift of concrete around turbine parts, it was required that it be placed in such a manner that there would be no mechanical vibration within five feet of turbine parts, and not more than one foot differential in the fresh concrete against the embedded turbine part. A constant stream of cooling water was applied to the exposed side of the turbine part during the placing of concrete and for fourteen days thereafter.

It is essential in erection of the rotating parts of the unit that the vertical axis of the generator and turbine coincide. The initial step in accomplishing this is in the factory check of the assembled generator shaft and upper section of the turbine shaft. No equipment was available in the United States of sufficient size or rating to check the thrust collar in place on the assembled turbine and generator shaft. It was, therefore, necessary to accept the manufacturing tolerance of the thrust collar, even though it had to be removed from the generator shaft for shipment. This tolerance is $\pm$ .0015" at the outer edge of the thrust collar face.

The General Electric spring-supported thrust bearing is being installed in the first twelve units. In this bearing each of the 12 babbitted segments is supported by a nest of 170 pre-stressed springs. The alignment of the thrust bearing has been accomplished by leveling the thrust bearing bracket to the point where the axis of the assembled generator and turbine shaft is vertical and straight, and is common to the axis of the turbine guide bearing. It has been possible to adjust the generator bearing bracket so that the shaft alignment tolerance is on the order of $\pm$ .0015". A rotational check has not been made of the McNary thrust bearings. The proof of a well-aligned thrust bearing is considered to lie in the movement of the thrust bearing shoe after the unit is in service. A special device which has been assembled in each bearing indicates the vertical movement of the thrust bearing shoes. A movement of .002" is considered excessive. None of the McNary bearings have that great a movement.

The method that has been developed at McNary for assembling the generator coupling is of interest. The turbine is installed with the weight of the runner supported on a tripod with the turbine shaft approximately 3/4" low. The bolts are installed in the generator coupling with the 3/4" gap existing between the two faces. The generator is then raised on jacks until shims can be placed on the turbine tripod so that when the generator rotor is again lowered, there will be a gap of only .010" or .015" between the faces. The lowering of the generator rotor is very critical and extreme care is exercised to see that the axis of the generator shaft does not shift during this procedure. The generator bolts are tightened by slugging to close the last few thousands of an inch gap between the coupling faces. A hydraulic wrench is then used to stretch the coupling bolts the prescribed amount.

Very gratifying results have been experienced with the installation of main turbines and generators. Seven units have been put in service with no bearing or transformer failures up to this time. No realignment of turbine bearings has been necessary and only minor corrections have been made in controls to put the units in service and keep them on the system.

Special Construction Features

Leakage through Crib Cofferdam at Spillway Bay 13

In planning for the construction of McNary Dam, considerable discussion evolved around the question as to whether the cofferdams should be built of sheet steel cells or of rock-filled wooden cribs. From experience encountered with the 450 feet of crib cofferdam above and below spillway bay 13 that acted as the river leg of the main second step cofferdam, it was fortunate that this was the extent of crib cofferdam on the job. This crib cofferdam was a source of trouble from the time the cofferdam area was pumped out until it was removed. An excessive amount of leakage varying from 70 c.f.s. to 120 c.f.s. occurred, depending on the stage of the river, and no method could be devised to improve the situation to any great extent. The cofferdam was built in the dry and allowed to stand for several months before it was put to use. By this time the tongue and groove sheathing had shrunk and opened up considerably. This closed up to some extent as it became soaked, but never to the extent where it shut off the leakage. The sheathing was placed before filling the crib, a factor which contributed to the leakage.

The downstream section of the crib extended from the ogee section of spillway 13 to cell No. 20 of the downstream leg, a distance of 360 feet. It had a maximum height of fifty feet, with a width at the base of fifty feet. The base sat on the completed stilling basin slab. Timbers forming the crib interlaced to make 12 foot by 12 foot compartments. The river side upon which the sheathing was placed was vertical and the back side stepped down as the height of the rows of compartments was reduced. The top had a width of two compartments, with the compartments next to the sheathing, filled with silt, backed up by fine spalls in the adjacent compartment.

Water coming through the sheathing from the first high water period washed the silt through the spalls and as much as 23 feet was washed out of some compartments. Steps were immediately taken to replace the silt with a blend of gravel, sand and silt, with some hay and straw worked into it. This did not seem to affect the amount of leakage to any great extent. It was then decided to drive sheet steel pile adjacent to the spalls inside the compartments containing the impervious material for the purpose of keeping the fine material from washing out through the spalls. This steel extended from the top of the cofferdam at elevation 279 to the top of the stilling basin slab at elevation 228. The steel would have cut off most of the leakage had it been continuous for the 360-foot length of the downstream crib, but this was not possible because of interference by the cross timbers of the crib. However, it was the general opinion that the steel did reduce the seepage to some extent.

Several things were learned as a result of the difficulties encountered with the crib cofferdam. First, a crib cofferdam can never be much tighter than the sheathing itself. As an added precaution two layers of sheathing, one layer tongue and grooved with heavy tar paper between, should be installed. A high crib should be filled with rock before the sheathing is placed. The top of the McNary crib settled a foot or more and all the settlement was within the timbers, since the crib rested on concrete. This settlement, no doubt, crimped and cracked the sheathing that was put on prior to filling. Another important thing to keep in mind is that an impervious section of earth can not be built in a crib cofferdam for the reason that the impervious material or any other material acting as a filter can not be placed or compacted under the

cross timbers running through the crib in such a manner that the material will not settle away from the bottom of the timbers to some extent. The smallest amount of settlement under the cross timbers immediately becomes a path for water which becomes larger as material is washed out. The combined cost of constructing and removing the crib cofferdam would have been greatly reduced had it been built of sheet steel pile.

Failure and Reconstruction of Cofferdam Cell No. 52

On June 18, 1951, Cell No. 52 of the downstream leg of the main second step cofferdam failed completely. At the time of failure, the cofferdam was being overtopped with a flow in the river of 472,800 c.f.s. The inside pool was at elevation 272.4 and the tail water was at elevation 268. The cause of the failure was never determined, but two plausible explanations have been advanced. First, it could have been caused by a failure in the interlocks. The cell was founded on a steeply sloping river bottom (about 1 vertical to 2 horizontal), making it virtually impossible to compute the resulting interlocking stresses. Secondly, it could have been the result of poor contact between the piles and bed rock or from erosion of the bed rock which would permit loss of cell fill to create an arching-over condition that could cause failure by sudden collapse. The cell was 52 feet in diameter and consisted of 168 straight web sheet steel piles varying in length from 42 feet to 75 feet. The cell was filled with 8,000 cubic yards of silty sand and gravel, and topped with three feet of cap rock. The allowable interlock stresses for design purposes at the time the cell was constructed was 12,000 pounds per lineal inch of interlock. The allowable stresses now in use by the Corps of Engineers on similar cofferdams is 8,000 pounds.

The failure occurred on a falling river. As soon as the lower leg of the cofferdam was free of overtopping, work was started on closing the breach. Violent surges were coming through the breach from the action of the water through the low spillway bays. It would have been almost physically impossible to have placed a steel cell across the opening under this severe condition. It was therefore determined to first construct a ring dike around the upstream side of the opening which would serve two purposes. First, it would permit the unwatering of the cofferdam so that work could be resumed on the main dam and powerhouse structure; and secondly, it would have a dampening effect on the surges that were coming through into the cofferdam area.

The ring dike was built by end dumping from trucks that had access to the work area over the downstream leg of the cofferdam. The material used in the dike was a mixture of gravel, sand and silt with rock dumped on the outside to protect the fill against the surging water. The dike proved to be remarkably tight, considering that it was placed in 20 or more feet of water. Before the ring dike was completed, work was started on reconstructing the cell. However, a complete new design was devised which changed entirely the shape of the cell from that of the original one, and also greatly reduced the interlocking stresses. It was in fact a cluster of eight cells with the center circular cell centering on the center of the cell that failed. Figure 9 shows the reconstruction of the cell under way, and the extension to the ring dike that was built parallel and adjacent to the crib cofferdam. This extension of the ring dike was for the purpose of cutting off the large amount of seepage through the crib cofferdam. It was built and removed at the contractor's expense. One of the principal reasons for adopting the cluster design was to be

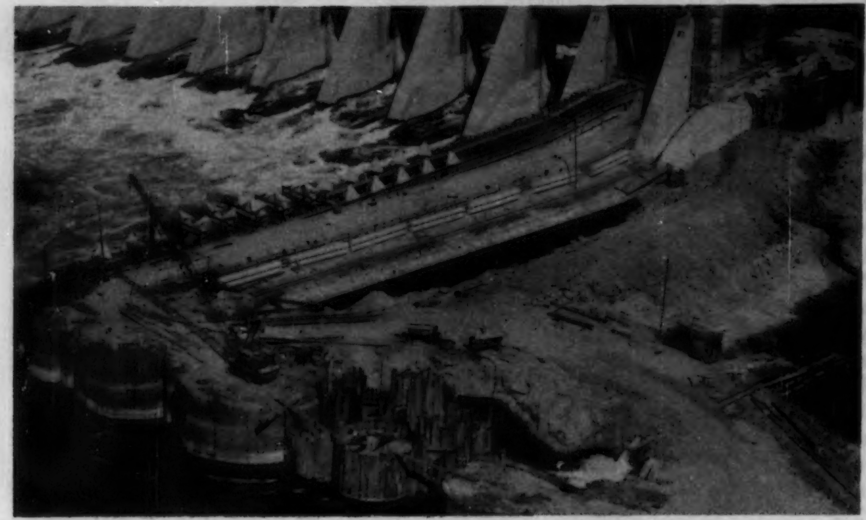

Figure 9. Reconstruction of Cofferdam Cell 52. Note ring levee and levee extension that controlled leakage through crib cofferdam.

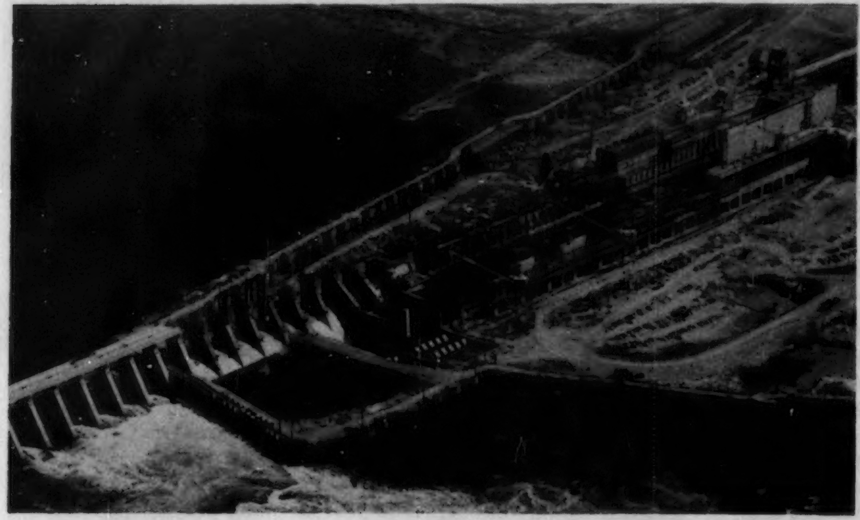

Figure 10. Auxiliary earth cofferdam showing portion of cofferdam to left of it that was removed prior to breaching the downstream leg.

able to get the steel in the outside perimeter of the upstream cells of the cluster in the clear of any of the steel from the old cell that might still remain on the bottom.

The contractor was able to construct the cell cluster without too much difficulty. However, some of the sheet steel piling on the outside perimeter of the upstream cells could not be driven through the fill of the ring dike to bed rock. This created some concern since the ring levee had to be removed sooner or later to allow for construction of the stilling basin and the stilling basin sill, the toe of which came within 28 feet of the cell steel that had not reached bed rock. A grouting program of considerable magnitude was undertaken with government forces in an attempt to consolidate the material inside the cells from bed rock up to a few feet above the bottom of the steel that had not reached bed rock. Since there was no way of determining the result of the grouting with any degree of accuracy, it was finally decided to disregard its stabilizing effect and leave in a considerable amount of the ring levee fill for the purpose of backing up the openings below the steel that were the result of inability to drive the steel pile to bed rock. In order to allow for leaving in a greater amount of fill to support the cells than would have been possible with only the 28 feet of clearance between the cells and the downstream toe of the stilling basin sill, the sloping downstream side of the sill was made vertical. The stability of this thinner, lighter sill section was assured by tying it into the 15-ft. stilling basin slab with heavy reinforcing steel.

Work on the dam was not retarded to any appreciable extent as a result of the failure. Although it cost the government $276,000.00 to close the breach, it did, however, serve as an object lesson for the design and construction of cellular steel cofferdams on rivers such as the Columbia. In the first place, cells should never be placed on steeply sloping rock without taking the precaution of leveling up the low side with concrete after the steel cells are set. If there is any likelihood that the rock, upon which the cofferdam is founded, will be subject to erosive action, concrete or bags filled with sand and cement should be placed in the bottom of the cells. A generous safety factor should be used in setting up the allowable stresses in the interlocks, since conditions are likely to arise when the cofferdam is put in operation that could not have been anticipated.

Repairs to Temporary Fishladders

Space does not permit a detailed account of the construction, operation and maintenance of the temporary fishladders during the period that the dam was under construction. They were quite extensive and costly to construct, maintain and operate. The original plan, which had to be modified greatly, called for temporary ladders through spillway bay No. 1 and spillway bay No. 13, and a third on the Oregon shore which paralleled the upstream leg of the cofferdam at a distance of approximately 100 feet, then made a 90-degree turn upstream to pass through the embankment section of the cofferdam. This temporary ladder operated only during flood periods when the second step cofferdam was overtopped. In order to enter this ladder, fish had to go over the downstream leg of the cofferdam and then through either the partially constructed powerhouse or the partially constructed south nine spillway bays. Figure 1 shows the location of the three temporary fishladders.

All temporary ladders were originally constructed of timber, but later the ladder through spillway bay No. 1 had to be replaced with a concrete structure. Extensive repairs were made to the one through spillway bay No. 13

Figure 11. View of entire project showing diversion through partially completed powerhouse.

with structural steel, steel piling, heavy timbers and counterweights. These two ladders had not been in operation long before it became apparent that it was a mistake to build a wooden structure such as these ladders in locations where they would be subjected to the attack of such violent water, particularly during high water periods. No serious difficulties were encountered with the Oregon shore fishladder since it was not subjected to the extremely severe action of the fast flowing water that tended to tear apart and destroy the two ladders through the partially completed spillway bays.

Temporary Settings of Upstream Miter Gate

The general plan of construction called for placing the navigation lock in operation at an early date, since work could not proceed to any considerable extent in the main river channel under phase three until the navigation lock was operative. The lock had to go into operation initially with only a very few feet of lift which necessitated the installation of a temporary upstream miter gate with a sill elevation of 258, sixty feet below the final sill elevation of 318. In order to eliminate the necessity of putting in temporary miter gates to span the entire 86 feet of the lock width, the upstream miter gate sill block was constructed initially with a 45-foot block-out in the center. This was the minimum width that would safely pass a 40-foot barge.

An intermediate or second setting of the temporary miter gate was required as a result of limiting factors which prohibited the raising of the pool in one operation to the minimum operating level of approximately 326 for the permanent upstream miter gate. The necessity for raising the pool to an intermediate level of 310 was due to a combination of conditions. The completion of some of the relocations and levees within the upper reaches of the reservoir was considerably delayed. The reservoir could not be raised to the minimum operating level of the completed lock until this work had been completed. It was determined that by the time the work could be completed, there would not be sufficient flow in the river to provide for the raising of the pool to the minimum operating level of 326 in one continuous operation without doing one of two undesirable things; extend the filling of the reservoir out over a very long period during which time the lock would be out of operation, or cut the filling time down to a few weeks by more rapid storage which would reduce the flow substantially below the requirements for the generating units at Bonneville.

The plan for raising the pool in two stages in conjunction with a second setting of the temporary upstream miter gate worked out very satisfactorily. Navigation was interrupted for a comparatively short period, and power production at Bonneville suffered only slightly, since the time and quantity of water required to bring the reservoir up during the low water period to the minimum elevation of 326 for operation of the lock was greatly reduced by having raised the reservoir to the intermediate level of 310 when there was ample flow in the river.

SUMMARY

The successful completion of McNary Dam and the placing of power on the line on schedule is due in great measure to the tireless effort on the part of the principal contractors to overcome tremendous obstacles in order to meet

the extremely tight construction schedule. Credit is also due the many smaller contractors and suppliers who made a special effort to complete their work on schedule, regardless of the labor and material shortage that existed during the critical period of construction as a result of the Korean War. The excellent teamwork and cooperation that existed between the Government engineer and the contractors deserve particular mention.

McNary Dam was built under the direction of the Walla Walla District of the North Pacific Division, Corps of Engineers. Colonel Louis H. Foote is the present Division Engineer; Colonel A. H. Miller, the present District Engineer. Valuable contribution to the preparation of this paper was made by Mr. R. L. Earnheart, Project Engineer, McNary Dam.

[illegible]

[illegible] the North Pacific [illegible] Colonel A. R. Miller, the present District Engineer, [illegible] by Mr. H. L. [illegible], Project Engineer, McNary Dam.

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JOURNAL
POWER DIVISION
Proceedings of the American Society of Civil Engineers

McNARY DAM - COORDINATION OF PROJECT DESIGN AND CONSTRUCTION

Otto R. Lunn[1]
(Proc. Paper 951)

On large, multi-purpose dams like McNary on the Columbia River, the design of the dam structures must of necessity be closely coordinated with construction. Such coordination is commonly referred to by names such as planning, scheduling, step-construction, phasing, etc.

In order to come to grips with overall scheduling of a large project the factors actually controlling the schedule must be clearly defined. At McNary Project, the basic factors controlling overall scheduling were:

1. Navigation kept open during construction.
2. Frequency and magnitude of annual floods.
3. Fish passing facilities provided during construction.
4. Power on the line as soon as possible.

These factors are actually design requirements and construction requirements must yield to the design requirements, if at all possible.

Navigation

To pass navigation during construction it was apparent, even in the early planning stage of McNary Project, that the lock should be built first, and that the lock chamber with a low upstream sill, then could pass navigation after removal of 1st step cofferdam.

Much effort was expended in scheduling for minimum closure time for navigation during the change-over from open river navigation passing McNary damsite during construction to temporary navigation through the partially completed lock chamber. One of the controls for the change-over was the upstream pool elevation which would permit navigation to pass from the lock chamber to the upstream navigation channel. This interim pool elevation was possible only with low upstream lock sill, temporary upstream wood miter gate, low spillway bays, gate slots in upstream spillway pier-noses, proper height of 2nd step cofferdam and appropriate designs of temporary fishladders which had to operate properly after the upstream pool was raised

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18 feet at low river flows. It must also be realized that after correct planning and design, the actual construction must be timed and accomplished exactly as specified or the battle with the Columbia River would be lost. Two conditions developed during construction which endangered the success of the battle. One was construction difficulties with eight rock-filled wooden cribs designed and constructed by the contractor. A delay of three weeks was occasioned thereby, which could have proven fatal to the successful closure of the main cofferdam. The other unfavorable condition that developed was an unseasonal rise in the river flow during October-November 1950 from the normal flow of 100,000 c.f.s. to 150,000 c.f.s. However, neither the three weeks delay nor the relatively high flow during the critical river closure, prevented the accomplishment of meeting the overall schedule and the closure time for navigation amounted to only 60 days (Mid-October to Mid-December 1950).

Annual Flood Flows

The effect of annual flood flows in Columbia River on the overall construction schedule is axiomatic to those engineers familiar with the characteristics of the river and with dam construction problems.

Analysis of past annual flood flows and of cost of cofferdams, demonstrated that cofferdams could not be economically built to heights that would prevent overtopping during annual peak flows. The compromise height of cofferdams appeared to result in flooding for one to three months each year, thus reducing the available construction time within the cofferdams to nine to eleven months per year. The most convenient tool for this determination was an "average" hydrograph which showed the envelope of peak flows on any date, the envelope of minimum flows on any date and the average flow curve for each date. The latter curve peaked at 500,000 c.f.s. and showed more than 400,000 c.f.s. from 15 May to 15 July, ie. two months. Trial designs and hydraulic model tests showed that a main second step cofferdam with upstream leg at Elevation 292 would protect the work area against flows up to 400,000 c.f.s. and this cofferdam height was adopted, built and performed satisfactorily. It should be noted that for scheduling purposes the "non-work time" inside the cofferdam is more than the apparent "net flooded time" would indicate. This is a result of time required to evacuate contractors plant and to flood the cofferdam prior to actual overtopping and the time required to pump the cofferdam out. In the case of McNary Main Cofferdam (45 acres inclosed) it took two weeks for the first time pump-down (Jan. 1950) while a subsequent pump-down consumed a less time and the last pump-down (July 1952) was accomplished in a few days. Steel sheet pile cells for cofferdams tighten up against leakage to a remarkable degree in a few months time, apparently from a combination of siltation and rust in the interlocks.

For the first step cofferdam at McNary and for certain auxiliary cofferdams it was possible to build higher than for 400,000 c.f.s. and a frequency curve for flood flow occurrence was used to judge the calculated risk of being flooded. The first step cofferdam was overtopped by the second largest flood of record, the now famous 1948 flood on the Columbia River. Damage to the cofferdam was minor and the construction schedule was not delayed.

In the case of the Oregon Shore Junior cofferdam no overtopping occurred, but the 1950 flood equalled the height designed for. It does not often happen

that an uncertain factor like the height of a cofferdam is assumed exactly right for a particular flood season.

Temporary Fish Passage

To a designer, temporary fish passing facilities spell difficulties. The need for such facilities was obvious at McNary and the situation for fish passage from the time of second step cofferdam closure until operation of permanent fish ladders was grim indeed. This period was long, from 17 November 1950 to 6 November 1953, or three years. The difficulties stemmed from the necessity of squeezing the 3,000 foot wide Columbia River to a temporary width of 600 feet, thereby creating a drop of 18 feet to 30 feet through the 12 low spillway bays each 50 feet wide. To be workable the fish passing facilities had to be located at the sides or edges of this narrow gap which meant in very fast, turbulent water.

The temporary fish ladders were designed as wood frame structures and rock-filled wood cribs. They were bolted together and appeared very substantial—on paper. They were also relatively inexpensive. However, during the first flood season, it soon became apparent that the fast, turbulent water adjacent to the temporary wood structures set up vibrations that gradually deteriorated the structures. Expensive, remedial measures had to be taken and successful passage of the fishruns was accomplished. The overall schedule was not delayed, but the margin of safety for accomplishing this was uncomfortably small.

The lesson learned at McNary in this connection is that no skimping/in design of temporary structures is permissable when subjected to the violent action of a major river in flood and that a timber structure is not suitable adjacent to deep, fast, turbulent water unless it is protected by a steel sheet pile face which must be attached to the structure in a very solid manner indeed. This construction experience will be reflected in more suitable designs of temporary structures in the future.

Power on the Line Requirement

The requirement of power on the line as soon as possible at McNary Dam developed after the preliminary scheduling had been done. McNary Dam was authorized a few months before the end of World War II and predictions at that time of the power situation in the Pacific Northwest for the next five years were for a surplus of power after cessation of hostilities. This prediction proved to be false. By 1947 the power demand in the Pacific Northwest hit the war-time peak level and has been climbing rapidly since. The overall scheduling of McNary Project was revised and accelerated, from one unit on the line every four months to one unit on the line every three months affecting Units No. 3 to No. 14, thus completing installation of 14 units one year earlier than scheduled. At the present writing eight of the 14 units have been put on the line according to the accelerated schedule and since these units have been operated almost continuously at rated capacity plus 15 per cent it is apparent that the need for McNary power was and is very great indeed.

In addition to the above four factors controlling the schedule other factors influencing the schedule must be given proper consideration, such as: evaluation of volume of work possible to accomplish between flood seasons,

non-interference between contractors, time loss because of winter freeze-up, possible rate of delivery of turbines and generators (manufacturer's shop capacity often controls this possible rate) and evaluation of time required for administrative actions. A final handicap to realistic scheduling on large Government projects is the uncertainty of yearly appropriations.

With the requirements of scheduling established the implementation of scheduling leads into design of structures—particularly structures such as cofferdams and fish passing facilities—where the construction needs are controlling.

In order to assure scheduled completion of powerhouse structure longer construction time was required than would be available after completion of the main second step cofferdam. A relatively small "Junior" cofferdam was therefore built on the Oregon shore in 1950 enclosing powerhouse assembly area, two main unit areas and area for the large pumphouse for fish attraction water. The construction of the junior cofferdam gave a one year earlier start on the powerhouse and permitted a more realistic scheduling of the large volume of work to be accomplished inside the main second step cofferdam during 1951 and 1952.

In the case of temporary fish ladders which were built of wood, the partial failure during the first flood season, made redesign and reconstruction necessary. The redesigns were made to fit the construction conditions and utilized available materials, but the added work the contractor was ordered to do, constituted a superimposed workload on an already tight schedule. Special efforts by contractor and engineering personnel again prevented delay in the overall schedule.

At a damsite with characteristics as at McNary, it was not advisable to have contractors bid on their own designs for cofferdams. The reason for this was the innumerable limiting factors involved (which the bidder could not be expected to know) and the sheer magnitude and risk of the cofferdam work, including a difficult closure. The cofferdams were therefore designed by the Government and unit price bids were received. In the case of the second step cofferdam closure, the Government engineers directed the work and were responsible for the success or failure of the closure operation. The success of the closure by means of precast concrete tetrahedrons weighing 12 tons each was most gratifying to the Corps of Engineers.

An important utilization of a realistic schedule is for predetermination of yearly funds requirements one and a half years ahead of receipt of funds. While adjustments in funds requests can be made as late as six months before final approval as to amount, the orderly processes of Government administration suffer from last minute changes. Furthermore, a job that is scheduled for several years ahead cannot change pace without economic losses.

On McNary there were about 75 construction contracts and 185 supply contracts each of which had to be fitted into the overall construction schedule and then scheduled individually. The contract schedule covered the period 1947 to 1956 and bracketed the periods of Controlled Materials Plan, Korean War and the steel strike in 1952. In spite of these serious handicaps to meet any kind of schedule, various steps and expedients were made to meet the overall schedule and eight McNary power units have been put on line on schedule.

The measures taken to keep McNary project on schedule involved cofferdams within cofferdams, use of powerhouse intake gates without hydraulic operating equipment, shortening of contract erection time by simultaneous

work by several contractors on power units, some substitution of materials (when specified material were unavailable or temporarily prohibited) and a concerted effort to co-operate with and get co-operation from the contractors.

In reviewing the actual construction record of McNary project it becomes apparent that the field construction work accomplished between the spring of 1947 and the spring of 1948 could readily have been telescoped in the period between spring of 1948 and spring of 1949. However, the funds made available for Fiscal Years 1947 and 1948 were insufficient for large scale operations and it was decided to make a start on actual construction with the funds available for Fiscal Year 1948. Consequently when construction time for McNary is counted from April 1947 to April 1954 we think in terms of seven years time, while actually a six-year schedule could have been accomplished as readily, provided sufficient funds had been made available at the proper time.

On diagram No. 1 the monthly costs are plotted. It is noted that two years averaged above four million dollars per month and four years averaged above three million dollars per month. The variations in monthly expenditures are caused by many things, the seasonal variation in construction because of flooded cofferdams (June-July) and winter freeze-up (December-February) being only part of the variation. The maximum month, July 1953, was caused by a change in bookkeeping, from "costs" to "accrued expenditures." However, the trend of the curve as represented by the average curve, is a useful tool to use for scheduling future jobs such as John Day Project on the Columbia River.

On diagram No. 2 the record of pouring concrete is plotted. While the mass concrete production for the lock and 13 bays of the spillway are impressive, the results obtained for the period 1 December 1951 to 1 May 1952 were truly amazing and exceeded the scheduled rate. This concrete was heavily reinforced powerhouse concrete and a steady rate of 100,000 cubic yards per month was maintained with a six-day per week program. This rate of progress may be considered the maximum practicable rate for large reinforced concrete powerhouses and should not be compared with the rate of placing mass concrete elsewhere.

To fit the schedule of supply contracts into the overall schedule, requires special skills in visualizing needs and erection conditions for cranes, valves, turbines and generators. The lead time on turbines, generators, governor equipment and transformers amounts to several years and while a theoretical schedule can be worked out, so many things occur to upset such a schedule. At McNary each of twelve 70,000 KW generators furnished by General Electric Co. constituted 70 railroad freight cars, shipped from Schnectady, N. Y. to McNary, Oregon. Naturally, time and order of arrival cannot be made to fit the actual detailed erection, so rehandling and short time storage is required. It was found that old flat cars, acquired by the Government solved the problem of temporary storage, because heavy parts could be transferred to the old cars under the powerhouse cranes and later delivered under the cranes when needed for erection. Considerable spur trackage is needed for such car storage, but the method is practical.

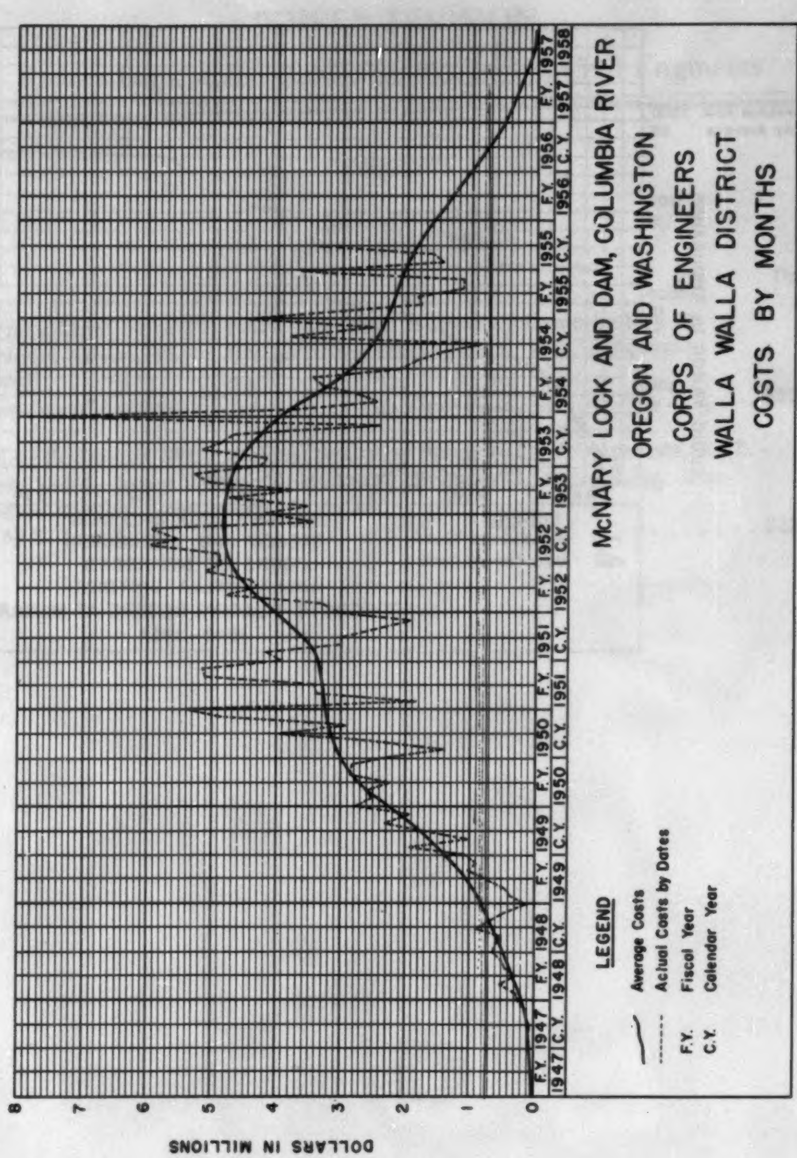
As previously mentioned McNary power installation schedule was accelerated from four-month intervals to three-month intervals. At the time it was believed that assembly space and available crane time were controlling factors and that any further acceleration of installation would require one additional powerhouse bridge crane and a temporary assembly area. Actual

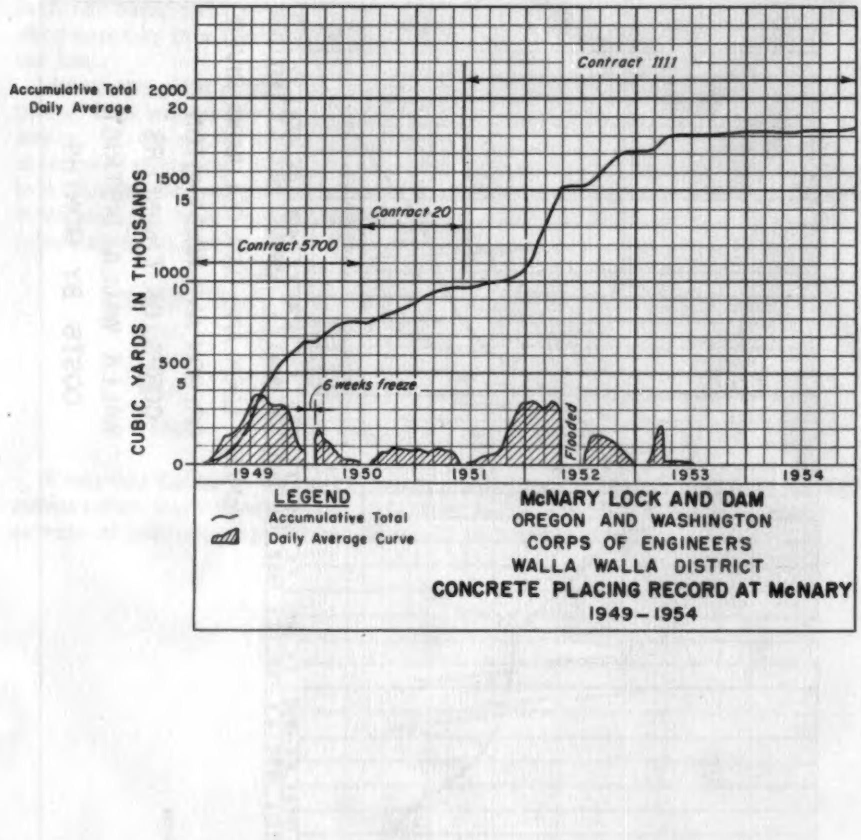
experience has now shown that crane capacity (time) and assembly space available at McNary were not the limiting factors, but that the shop capacity of the manufacturers and their ability to crate and ship fast enough were the limiting factors. Consequently, when competitive bidding results in one manufacturer being awarded all turbines or all generators, such a manufacturer's shop capacity is a limiting factor on how rapidly power units can be put on the line.

From the point of view of construction, McNary Dam is 97 percent complete. It is well at the end of a job to take a look at the performance as a whole. To the writer the actual percent increase (contingency) of final payment over contract bid amount furnishes a figure which indicates the extent to which the job was properly planned, designed, constructed and administered. If we take the three major construction contracts at McNary Dam we get the following tabulation:

Contract No.	Bid	Final	% Increase
5700	$21,754,000	$ 23,678,000	8.9%
20	15,834,000	17,760,000	12.2%
1111	58,416,000	62,350,000	6.7%
Totals	$96,004,000	$103,788,000	8.1%

When this figure of 8 percent actual contingency increase on $100,000,000 construction work is compared with other large, heavy construction jobs—private or public—it appears to be very favorable indeed.


McNARY LOCK AND DAM, COLUMBIA RIVER
OREGON AND WASHINGTON
CORPS OF ENGINEERS
WALLA WALLA DISTRICT
COSTS BY MONTHS
LEGEND
Average Costs
Actual Costs by Dates
F.Y. Fiscal Year
C.Y. Calendar Year
DOLLARS IN MILLIONS
0
1
2
3
4
5
6
7
8
F.Y. 1947
F.Y. 1948
F.Y. 1949
F.Y. 1950
F.Y. 1951
F.Y. 1952
F.Y. 1953
F.Y. 1954
F.Y. 1955
F.Y. 1956
F.Y. 1957
1947
C.Y. 1947
C.Y. 1948
C.Y. 1949
C.Y. 1950
C.Y. 1951
C.Y. 1952
C.Y. 1953
C.Y. 1954
C.Y. 1955
C.Y. 1956
C.Y. 1957
1958



Accumulative Total 2000
Daily Average 20
CUBIC YARDS IN THOUSANDS
1500
15
1000
10
500
5
0
Contract 1111
Contract 20
Contract 5700
6 weeks freeze
Flooded
1949
1950
1951
1952
1953
1954
LEGEND
Accumulative Total
Daily Average Curve
McNARY LOCK AND DAM
OREGON AND WASHINGTON
CORPS OF ENGINEERS
WALLA WALLA DISTRICT
CONCRETE PLACING RECORD AT McNARY
1949 – 1954

JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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DISCUSSION
(Proc. Paper 952)

Note: Paper 952 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 2, April, 1956.

Discussion of
"PERMEABILITY, PORE PRESSURE AND
UPLIFT IN GRAVITY DAMS"

by Roy W. Carlson
(Proc. Paper 700)

SERGE LELIAVSKY,[1] M. ASCE.—Mr. Carlson's paper is, essentially, a "tour d'horizon" of a set of investigations on uplift in dams carried out recently, by a certain group of institutions and persons, which however, does not include the report on "Uplift in Dams," published, as late as 1951, by the Subcommittee specially appointed by ASCE to settle, once and for all, this problem.(1)

In fact, in this respect—paradoxical though this may appear—report and paper must be considered together, as a whole, for they are unanimous in exhibiting the same characteristic feature, viz. an attempt to bypass classical theory and reach the required result by other means; such for instance as the somewhat antiquated artifice which is described as "idealised concrete" on page 700-6 of the paper.

The number of oversimplified diagrams of this class— which, it will be remembered, were first introduced into the theory of uplift by Fillunger(2)—is legion. The danger of such over-simplifications is that an almost imperceptible alteration in the idealised model, reverses, frequently, the result.

Since, however, the author seems to be rather keen on the method, it would be of interest to know his reply to the problem proposed by Professor K. V. Terzaghi at the III Congress on Large Dams, in 1948,[2] viz. what happens with the uplift pressure, if we shift all the imaginary solids of the "ideal" pattern, so as to form a number of vertical columns; and all the voids so as to form vertical wells?

To cut a long story short, it will suffice to remember that idealised models of this description became obsolete when Terzaghi gave the differential equations which allowed to solve the problem in hand, as easily and definitely, as the elastic problem was solved in the beginning of the last century, by means of the differential equations of Lamé (and others); provided of course, they could be integrated. The point being that in addition to his experiments, frequently quoted in some of the publications on the subject, Terzaghi contributed, also, a general theory of uplift, which is essential in studying the problem. In view of his reputation, it is possibly surprising that his theory is, apparently, not known to the author of the paper, nor to the members of the subcommittee quoted earlier.[3]

Had the report of this subcommittee been published with a discussion, there is no doubt that its shortcomings would have been pointed out, and its value, thereby, enhanced. Since however, it was produced without discussion,

1. Civ. and Hydr. Engr., Cairo, Egypt.
2. See Proceedings of the Congress.
3. The writer resigned from the Subcommittee in question, owing to the attitude taken by its members when he suggested that without Professor Terzaghi, their findings could not be final.

it attracted little attention and became soon forgotten. The fact itself, that Mr. Carlson's paper has been written and published so soon after the Subcommittee's report, shows that the Subcommittee were not successful in their attempt to settle the problem, which is to be regretted for its members were outstanding engineers whose names were well known to the earlier generations of dam designers.

It seems, therefore, only logical to suggest that should further progress in this branch of engineering science be aimed at, the publications of Foppl, Fillunger, Rudeloff and Panzerbieter, Hoffman, and in particular Terzaghi, must first be translated into English, and thoroughly assimilated.

Looking at the paper under review, from another angle, it may be surprising that at a time when so much attention is focused on the Concept of "ultimate strength" or "limit" design, the methods and experiments described by the author are, in fact, a step backward. Though the author does not acknowledge the point explicitly, as is usually done in modern technical literature, the method of the tests he describes is essentially the same as used earlier by the writer.(3) With that difference however, that whilst the writers conclusions were based on the "ultimate," or "limit," conditions, at which the test piece broke under the effect of uplift (i.e. of the interstitial pressure of the water injected in the concrete), the author's results are derived from the deformation, of unbroken specimens. It is suggested that the former method is the more correct and more advanced solution of the two.

The reader is asked whether, if he were required to verify the resistance of a bridge, he would base his conclusions on an average section of the girder, or on its weakest section? The reply is obvious, and the same applies to the uplift problem as well, for here as in a bridge, failure occurs at the weakest section; that is, that section in which the effect of uplift is the greatest, whilst the resistance is the lowest. This general principle defines the location and method of the surface of rupture. Therefore, any conclusions (whatever they may eventually be) based on an average section of the specimen—as was the case in the authors experiments--are valueless, except, possibly, as additional information throwing light on some secondary aspects of the problem, but not as basic material, for establishing design principles.

In this connection, an analogy with the main principles of reinforced-concrete design might be enlightening. It is a matter of common knowledge that the sectional area controlling the deformation of a reinforced-concrete beam is materially larger than that assumed in the calculation of resistance, because the former includes some part of the extended concrete also, whereas the latter is confined to compressed area only. The reason being, that deflection is an accumulated result of numberless infinitesimal deformations and depends, therefore, on average conditions throughout the entire length of the member; whereas failure occurs in one point only, i.e. the weakest section (as defined earlier), and it is this section, therefore, that must (and is) included in the calculation of resistance. The same principle is also valid as regards the uplift experiments, for here as in a reinforced-concrete beam, the observed deformation is symptomatic of average conditions (that is an average uplift area) throughout the entire length of the specimen, between the observation points. Whereas failure occurs at the weakest point, in which the percentage of the effective area is a maximum and the resistance a minimum, and the dam must be calculated accordingly, i.e. depending on the results obtained with specimens tested to destruction, rather than on the measured deformations of the unbroken test piece.

We know, of course, from experimental mathematics that recorded figures

based on one single section are more affected by accidental circumstances than averages, and this explains the reason which prompted the writer to use the mathematical theory of probability, in systematising the results of his numerous tests on the failure of specimens. Experience of that nature is believed to be rather important, for it allows a deeper insight into the physical circumstances of the failure, particularly when the latter is produced by interstitial pressure in combination with direct load. For instance, such phenomena as the "agony" period described by the writer,(4) could not have been obtained otherwise, and knowledge of this kind is certainly more valuable than speculation on "idealised" models.

Another fallacy incorporated in the argument of the author of the paper, is the implicit assumption that permeability and porosity, on the one hand, and the effective uplift area on the other, are, of necessity, correlated.

This is indeed, an error, for extensive practice with specimens tested to destruction tends to prove the existence of two different concepts: the micro-structure and the microfailure. Since the author of the paper places so much faith on his idealised models, another idealised pattern may be suggested—just for once—though not as a proof, but only, as an explanation of what is meant by the two quoted concepts. In this connection attention is called to Fig. 1 which shows two schematised materials with obviously different porosities; but, in both cases, the effective uplift area is 100 per cent if the failure surfaces, in between the pores, are vertical, and is 60 per cent if these surfaces are inclined.

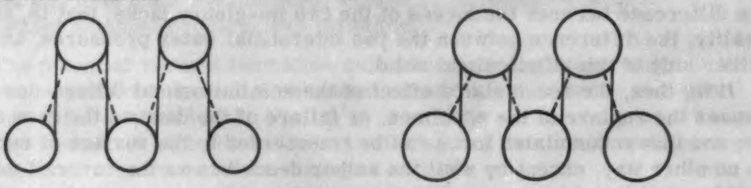

Figure 1.

Thus, depending on the inclination of the surfaces of failure, any effective uplift area is consistent with any porosity. The structure of the surface of failure is, consequently, the all-important consideration for determining the uplift area, and it is obvious that in order to obtain experimental information on failure, the test-piece must be made to fail.

As early as 1914, Fillunger showed that had the surface of failure been plane, there would have been no uplift force at all, because the additional weight of the water contained in the pores would have then balanced the additional interstitial pressure. That uplift does at all exist, and is, in fact, a significant parameter in designing dams, was explained by Fillunger himself, by the differences in the porosities of binding material and aggregate respectively, whilst Terzaghi propounded the much more convincing and better established principle, namely, that the porosity of the surface of failure was not the same as the average volumetric porosity of the material (and this, as a reasoned exception to Delesse's law). It follows that the effective uplift area cannot be determined from the deformations of specimens, which depend on the average porosity.

This instance is believed to show that before undertaking experiments which are intended as pioneer work, previous literature on the subject must be fully assimilated.

Another mirage the paper seems to be pursuing are the attempts to produce specimens of equal permeability. No matter what results may be ar-

rived at in the Laboratory, Mr. Mary[5] and the writer[6] have shown that in actual dam building, very wide differences may be expected even in samples taken from the same batch.

The 800-year curve in Fig. 1 is certainly sensational, but scarcely consistent with the fact that large quantities of percolating discharge develop frequently within a year after the dam is built. The logical chain:-

<u>discharge</u> - <u>velocity</u> - <u>gradient</u> - <u>pressure</u>

yields evidence that the least that could be said, is that the diagram in question, is misleading.

Apart from the features mentioned above, the paper abounds in sweeping statements about staggering discoveries made by others.

The writer regrets he cannot agree with these "discoveries". For instance, the author of the paper is certainly wrong in saying (see pp. 700-6 and 700-7) that it has been proved that there is no overall pull in the specimen tested in the manner first used by the writer,[7] and later by Davies, and that "the uplift is produced locally by the pore pressure acting as a jack within the pores."

Nothing of the sort has ever been proved. That interstitial pressure acts as a jack within the pores is, of course, correct, but since the infinitesimal solid between the pores is surrounded by the interstitial water on all its sides, we must imagine another jack being applied on the opposite side from the first jack, the net result being, therefore, an infinitesimal force equal to the difference between the forces of the two imaginary jacks; that is, in reality, the difference between the two interstitial water pressures, acting on either side of the infinitesimal solid.

It is, then, the accumulated effect of these infinitesimal differences which causes the rupture of the specimen, or failure of the dam as the case may be; and this accumulated force can be transferred to the surface of rupture in no other way, except by what the author describes as the "overall pull."

This instance supplies a vivid demonstration of what was said earlier on "idealised" models, viz. the ease with which such a model (in this case, the jack) can be made to prove the extreme opposite of its original argument.

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Discussion of
"HYDRAULIC DESIGN OF THE SANDOW PUMPING PLANT"

by R. T. Richards, E. T. Keck, and J. Junget
(Proc. Paper 948)

S. LOGAN KERR,[1] M. ASCE.—This portion of the overall project at Rockdale, Texas, is of particular interest since it represents the satisfactory solution of a large number of complex problems. The economic design of a pumping station with a long pipe line, the intermittent and variable flow subject to legal restrictions on the withdrawal of water, an unfavorable profile, the requirements for automatic regulation and potentially severe water hammer conditions, all had to be given the most careful study before the final design was settled.

As consultant to Ebasco Services, Inc., it was the writer's privilege to guide certain of the hydraulic studies, particularly the protection against water hammer and to assist in setting up and carrying out the field test program.

It is fortunate that the design conditions and the test results have been published and thus made available for study and guidance by others.

The water hammer problem was threefold.

1. The potential vacuum formation and subsequent pressure surge at the pump manifold since if was located some 50 feet or more above the low water suction level.
2. The numerous high points in the profile of the line itself, each having a potential vacuum and return surge condition.
3. The premissible rate of restarting the pumping units following a shut down due to power failure to avoid heavy surges when the various sections of the water column rejoined.

Dangerous surge conditions, due to the first two items, could be avoided on the line by the use of quick-opening vacuum valves, which admitted air to the line whenever the pressure dropped below atmospheric. The valves were designed to remain open until all the admitted air was expelled and water started flowing through them. The valves thus acted as relief valves which were already open to eliminate pressure rise.

The closing effort was due to internal pipe line pressure and the bouyancy of the float chamber. The rate of closure was regulated to limit surge pressures due to cutting off the relief flow.

The third problem was solved by limiting the opening of the check valves on the pumps to a safe rate of 4 minutes. The closing rate of 6 minutes was established to prevent any vacuum formation that would trip the surge valves.

The system was quite effective, reducing the surges to less than the design pressures of the line. Another paper(1) describes the tests in greater detail and shows the results with the surge control equipment in operation.

The question of running pumps in reverse to relieve surge pressures in the pumping stations following a trip out of the power supply has been the sub-

1. Cons. Engr., S. Logan Kerr & Co., Inc., Flourtown, Pa.

ject of a great deal of discussion. Many plants use this simple and effective method of relieving surges in combination with slow closing check valves without damage to the pumps or motors. If the manufacturer knows in advance that such service will be required, the seal rings, lock nuts, bearings, and driving motors are arranged to withstand sustained reverse rotation at or above normal forward speeds.

In the case of the Sandow Pumping Plant, one opinion stated that 20 seconds of reverse rotation would destroy the bearings. During the field tests, one unit was run in reverse, driven by a second pump. After some 45 minutes, no evidence of damage could be found. The writer feels that the reverse rotation could be continued for an indefinite time without damage to the pumps or their driving motors.

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JOURNAL
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: THEIR PHILOSOPHY

Andre Coyne,[1] Hon. M. ASCE
(Proc. Paper 959)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June 1956 at Knoxville, Tennessee.

Since the last symposium on masonry dams (April 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, two papers have been approved: "Arch Dams: Their Philosophy," (Proc. Paper 959) by Andre Coyne, Hon. M. ASCE, and "Arch Dams: Trial Load Studies for Hungry Horse Dam," (Proc. Paper 960) by Robert E. Glover, M. ASCE, and Merlin D. Copen.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

The Past

There is nobody who can flatter himself on being the inventor of the arch dam, and there is quite a chance we will never discover how the whole setup started.

The first historical specimens are to be found in Spain and Italy, a century apart.

In Spain, Almanza and Elche dams date from the 16th century. In Italy, Pontalto dam (Fig. 1) built in 1611 and raised several times since then, was mentioned by the late lamented Noetzli. The similarity of Pontalto dam with the bridge which is above it, is so obvious that it hardly needs to be pointed out, and it illustrates, better than any highbrow argument, how an arch dam is a kind of bridge overturned in an upstream direction. The oldest known examples are just like this and if the comparison seems a bit far-fetched today

Note: Discussion open until September 1, 1956. Paper 959 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 2, April, 1956.

1. Inspector General, Cons. Engr., A. Coyne and J. Bellier, Paris, France.

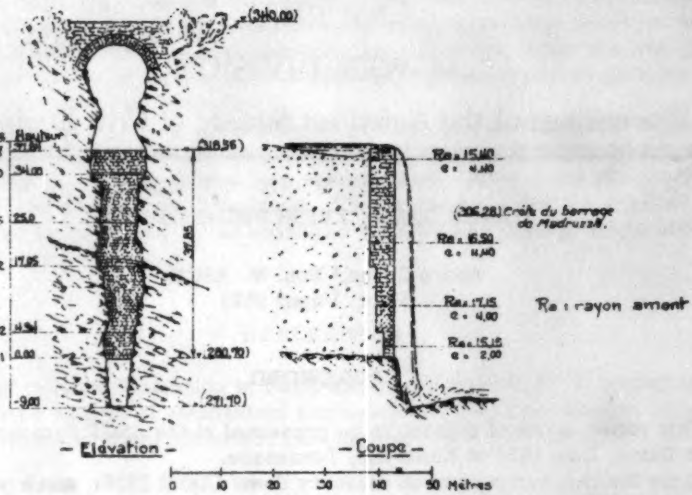

Fig. 1
Barrage de Pontalto

Fig. 2
Roman Bridge
(Photograph by: Ollière, La Mure)

it is because we vary the thicknesses and curvature so as to try and adjust our arches to the shapes of the valleys and the thrust of the water.

In France, the ancestor of arch dams is Zola dam near Aix-en-Provence, built in 1843. Zola, who designed it in 1839, was the father of the famous novelist. He really designed two dams, one 30 m (98 ft) high and the other 48 m (157 ft) high, on the same basic principles, but died before seeing his projects realized. Only the first dam was built, to a height of 36 m (118 ft), after his death.

At that time massive walls were the only known solution. Their dimensions were worked out by rough and ready empirical methods without taking any account at all of lateral support. The unquestionable merit of Zola was the fact he was daring enough to count upon an arch effect resulting from a curved setting out. Accurate calculation of the arch effect was beyond him - and of course beyond the technical knowledge of his times - but in his notes he underlines the safety given to his dams by their curvature and by the strength of the banks. He thus justified himself, against Academic criticism, for using lesser thicknesses than those admitted as standard in these days.

Then it is only just to mention in this historical review, the American pioneers who, a little bit nearer our days, had the courage of constructing arches, lots of arches. The most daring of these are, curiously enough, the oldest: Bear Valley and Upper Otay.

The great Jorgensen and his arches with a constant angle deserve special homage, for all modern Engineers are aware of having drawn their inspiration from his works.

Whence the Strength and Safety of Arch Dams?

The comparison just made of an arch dam to a bridge overturned in an upstream direction needs two explanations.

Firstly, contrarily to what happens in bridges, the weight and the load, instead of acting on the same plane, act more or less at right angles in dams.

Then, this time as in bridges, the thrust reactions of dams, and generally speaking their stabilising stresses, increase with the load. Contrarily to what happens in gravity dams, it is the structure itself that acts, producing practically automatically its own reactions and stresses, right up to the point strictly necessary for maintaining equilibrium.

This is why it has been said that arch dams work as self-sealing plugs, becoming stronger and more taught as the thrust of the forces bearing down on them increases. In hard fact, the total and general crushing of their constituent materials is their only yield point.

It goes without saying however that this faculty works in different ways, for arch dams are hyperstatic structures: even more so than encastered bridges because of the number of their peripheral connexions.

In other words, arch dams find many different ways - perhaps even an infinity - of solving the stability and strength problems required of them. There is on the one hand the inevitably unpredictable behavior of the foundations, where accurate advance estimates are difficult and on the other, the continually changing conditions created by different heads of water and above all varying temperatures. Arch dams, like the Roman bridge which to this day overarches the River Drac in France (Fig. 2) deal with all this by drawing on their hyperstatic reserves.

How then can we be surprised that such a simple principle has never betrayed the confidence placed in it - perhaps instinctively, and that no accident has been seen as yet in a type of structure which actually seems to be invulnerable?

What is more, it was enough, in the case of a structure justly reputed vulnerable, like Ternay gravity dam (Fig. 3), to give it a slight curvature (radius 400 m (1310 ft)), to save it from destruction. This without any research or calculation. The fact that it was the curvature alone that saved Ternay is all the more certain when we realize that all the other dams with the same cross-section but a straight setting-out have failed as a result of uplift.

There is however one failure of an arch dam on record. It was the one we built for experimental purposes, precisely so as to test it to failure (Figs. 4 and 5).

It was a simple concrete arch 3 m (10 ft) high, 20 m (66 ft) downstream radius and only 20 cm (8 in) thick, encastered at each end in excellent quality rock. Its upper part was covered over with a concrete roof with a watertight device. Thus was created a closed space in front of the cliff, where pressure could be built up to the limit tolerated by the arch. After several progressive tests when various measurements were made, a test to failure was applied. Failure started where the arch was thicker than elsewhere, and gradually extended, whereas in the parts of normal thickness, the concrete worked at about 300 kg/cm2 (4,300 lb/in2) without showing any visible signs of excessive strain. So the experiment revealed both the great strength of arches and the drawbacks of any awkwardly applied stiffening which, far from strengthening structures, is a cause of weakness. Elasticity is the essential quality of a good arch.

Apart from this exception, the conclusions which were valid twenty-five years ago, when Noetzli, with a keen sense of prophecy and perfect comprehension of his subject, founded his Study-Group for Arch Dam Investigation, are still valid today.

There is no known failure of an arch dam.

Evolution of Basic Criteriums

Since Noetzli, however, the criteriums applied have undergone a notable evolution, specially as far as the working stresses applied are concerned.

About twenty years ago, the average stresses, calculated by the tube formula for the great prototypes: Pacoïma, Diablo Dam, Ariel Dam, Marèges, Santa Luzia, were about 25 to 30 kg/cm2 (400 lb/in2). Today, they run up to 50, 60, 70 kg/cm2 (700, 850, 1,000 lb/in2) at Rossens, Val Gallina, Tignes, Salamonde, La Palisse, Malpasset, etc. (Fig. 6).

Exceptionally, average stresses exceeding 100 kg/cm2 (1400 lb/in2) were applied in the case of le Gage Dam.

With the average theoretical stresses, effective compressive stresses have risen from 30 to 40 kg/cm2 (400 to 550 lb/in2) (Marèges) to 60 and 70 kg/cm2 (850 and 1,000 lb/in2) (Salamonde) and even 110 or 120 kg/cm2 (1550 and 1700 lb/in2) at le Gage. A few words on this record dam will not be out of place here.

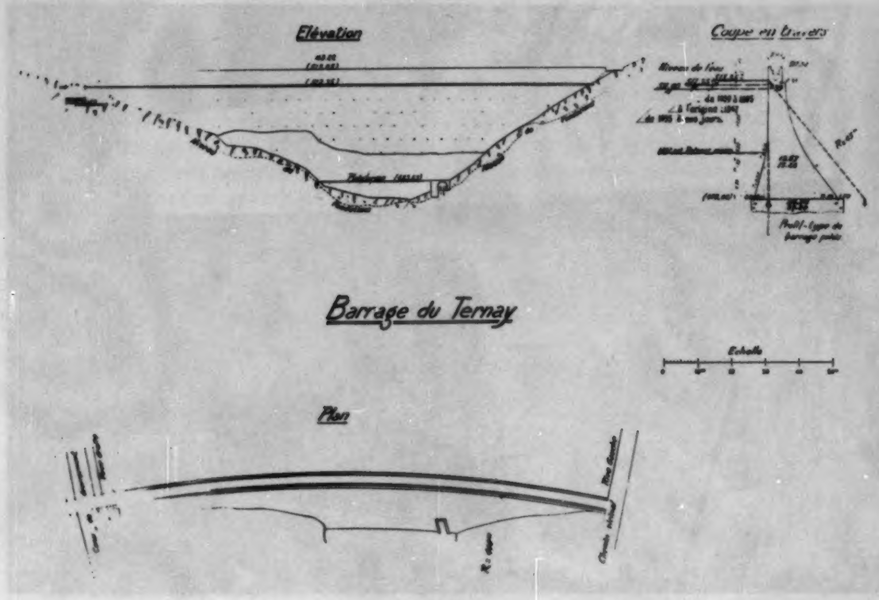

Fig. 3
Barrage du Ternay

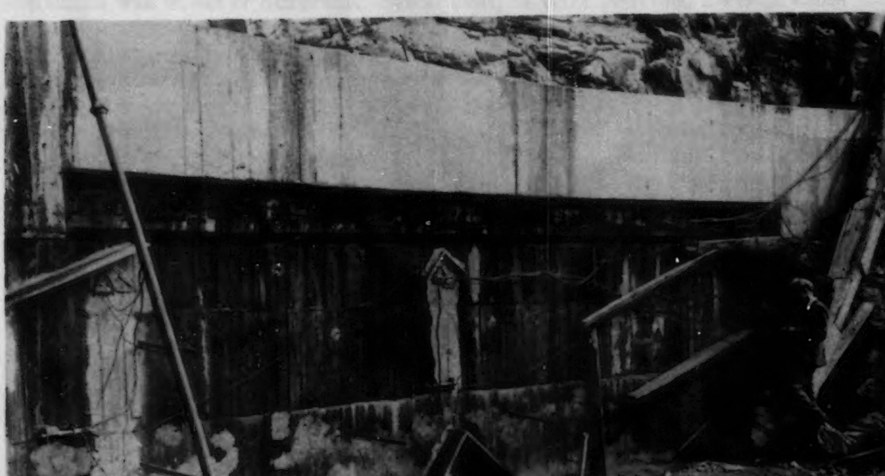

Fig. 4
Experimental arch dam before test to failure.

Fig. 5
Experimental arch dam after test to failure.

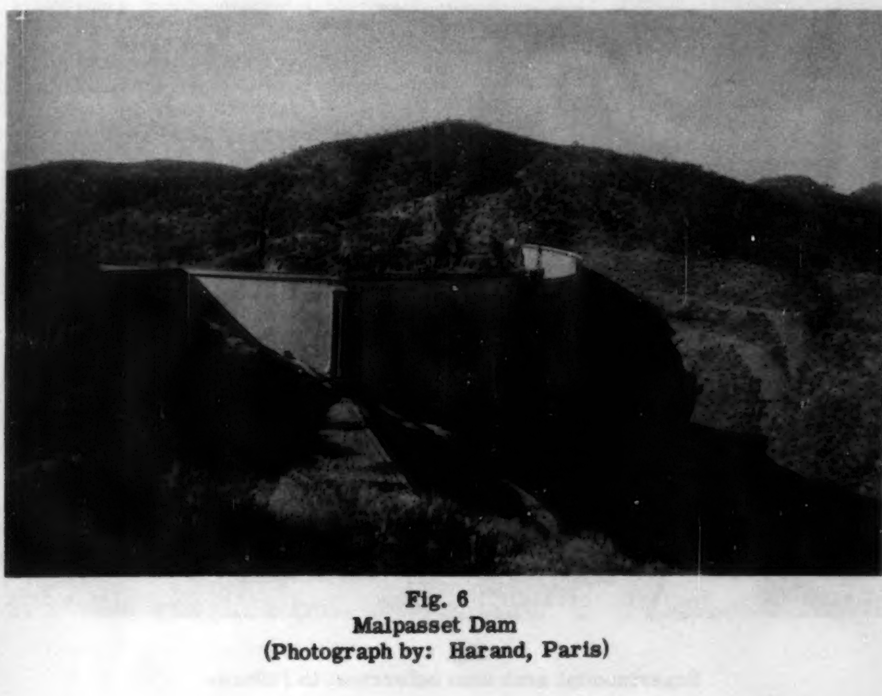

Fig. 6
Malpasset Dam
(Photograph by: Harand, Paris)

Le Gage Dam

A very thin arch, 38 m (125 ft) high (Figs. 7 and 8) built on one of the higher tributaries of the Loire in the framework of the Montpezat scheme, where water coming from the upper part of the Loire catchment area which would normally have flowed into the Atlantic Ocean, was diverted into a river running into the nearby Mediterranean. The object of this operation was to secure higher and more direct head. As the location was suitable, le Gage was made a sort of experimental dam, by designing it with the exceptionally high stresses mentioned above.

The first practical consequence was a reduction, also exceptional, of concrete volume for the dam. It is only 18/100 of what it would have been for a gravity dam at the same site, with the same excavation depth. We all know that for modern arch dams the proportion is still of the order of 1:3, i.e. twice as high as at le Gage.

The reduction in volume thus realised secured all the more economy because the working site of le Gage was only a few kilometers from the larger dam of La Palisse, then under construction. So as the quantities required were small, ready-mixed concrete was transported by road to the smaller dam. No question of borrow-pits or special mixing plant and the resulting expense. Over and above the economy in concrete unit-prices, there was a considerable saving of time. The dam was completely constructed within a single summer season.

Deflections and stresses were of course frequently measured when the reservoir was filled. It was thus ascertained that they were at no point above the maxima expected by the designers, but that the localisation of these maxima was however different. Better still, le Gage provided a remarkable example of the variety of ways arch dams have at their disposal for solving the elastic stability problems they are up against. Measurements have for instance shown that on filling the reservoir the dam foundations underwent very marked changes resulting from local subsidences of the rock, of excellent quality however on the whole. The distribution of stresses altered several times running and in some places the concrete freed itself from excessive extension by cracking, without any serious consequences.

It is thus not surprising that the true behavior of the structure was markedly different from that foreseen in the calculations, even those made on the basis of the most complex methods of adjustment, for the assumptions did not correspond to the actual conditions, which moreover, varied as the filling of the reservoir went on. In last analysis, it would seem that the fairly simple method of plunging arches gave the most satisfactory forecast of stress distribution in this arch.

Evolution of Shapes of Arch Dams

Parallel to the progress realized in stress values, shapes have thinned down, and have, further, evolved in very different directions.

Sometimes the lower part of the arch overhangs on the upstream side, a method applied for the first time twenty years ago at Marèges. This dome shape is the most suitable one where extension has to be fought, particularly at the encastrement of central cantilevers as in the case of Salamonde (Fig. 9), Cabril and many Italian arch dams.

Sometimes the arch slopes in a downstream direction as for Enchanet and Couesque (Fig. 10), so as to reduce the radius of the lowest part, deliberately

Fig. 8. Le Gage Dam
(Photograph by: H. Baranger, Paris)

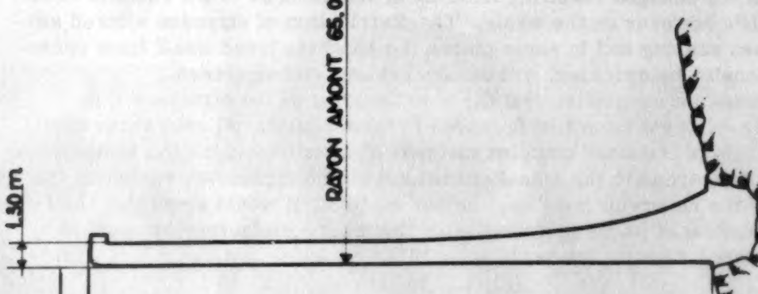

Fig. 7
Barrage du Gage

Fig. 9
Salamonde Dam

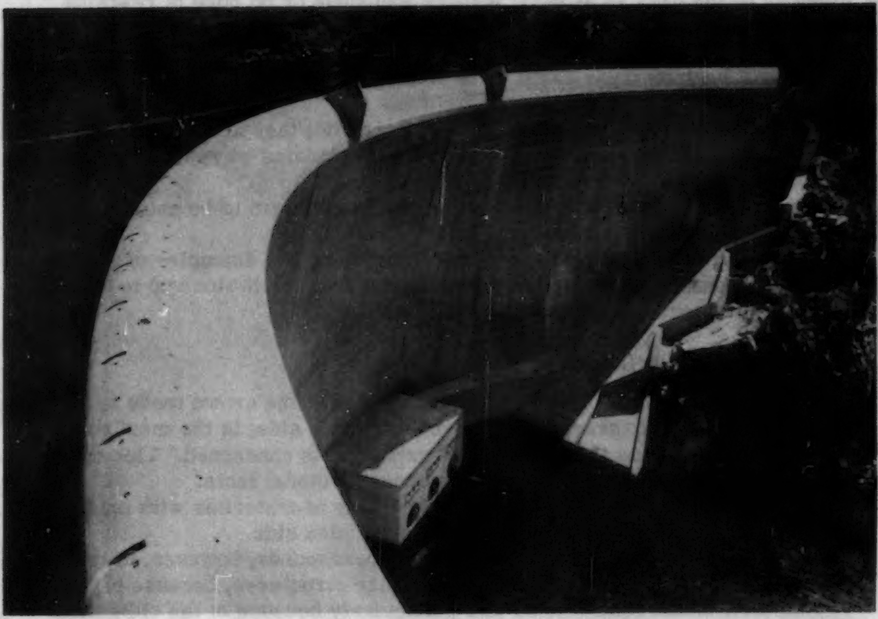

Fig. 10
Couesque Dam
(Photograph by: H. Baranger, Paris)

accepting breakage of the bottoms of the central cantilevers, which results in a crack subsequently plugged by grouting, as at Enchanet.

Shapes are sometimes simplified to their ultimate limit, for facilitating construction, as at Pont-en-Royans; for reasons concerning the foundations, as at Bioge (Fig. 11) and la Mandraka; or for the spillway, as in the case of Grangent, with its 5000 m3/s (178,000 cf/s) capacity crest spillway.

Then on the contrary for other reasons but still with the same objective of economy, we have complications of structure, such as local stiffening of the arch by adjoined or incorporated appurtenant structures (intakes or outlets); or else the elastic deformations are thwarted by blocking the toe of the dam with an appurtenant structure or even a large powerstation carrying a ski-jump spillway, as at l'Aigle (Fig. 12), Saint-Etienne-Cantalès and Chastang; or else a large opening is made right in the middle of the stress-flux, in order to provide a bottom sluice so as to eliminate temporary diversion tunnels as at Marèges and l'Aigle or to serve as a flood evacuator as at Castelo do Bode, Chastang (Fig. 13) and la Roucarié. Or else, even the mass itself is hollowed out and the power house placed in the orifice as at Monteynard.

Finally, as growing success lead to even greater daring, and progressively justified itself, arches were built even on locations which would have been considered unsuitable not so very long ago.

As at Bin el Ouidane, great asymmetry was accepted. Suspect foundations were tolerated with of course due precautions if the ground was hollow: grouting, as at Castillon; if the banks appeared unstable: strengthening with cables as at Castillon and la Chaudanne; if the ground was relatively soft: widening of the base as at Bort; if there was little or no hope of reaching sound rock at an economic price: deepening of the underground part of a thrust block, as at la Mandraka.

Looking back on these dams, it is clear that they are very different from each other.

They have however one common characteristic; they are all arches.

There is no doubt whatsoever that this fact explains why they hold out so perfectly.

Because an arch is curved, the difficulty would seem to be not to make it hold up, but to knock it down.

Serious accidents do not even occur: there are two examples of arch dams where a buttress was washed away, while the arch itself stood up to the pressure of the water: Moynie and Lake Lanier dams.

Economics of Arch Dams

A simple adjustment of the radial deflections at the crown tends to show that the dome-shape, overhanging on the upstream side, is the most rational one, at least as far as the thwarting of extension is concerned. This conclusion is immediately confirmed by membrane or model tests.

It is this shape that gives maximum economy of materials with maximum economy of stresses, particularly on the extension side.

Maximum economy of materials rarely corresponds, however, to true economy, specially for small or average size structures, because of the special formwork requirements and particularly because of the obligatory slowing up of construction through complicated shapes and layouts or through excessive thinness.

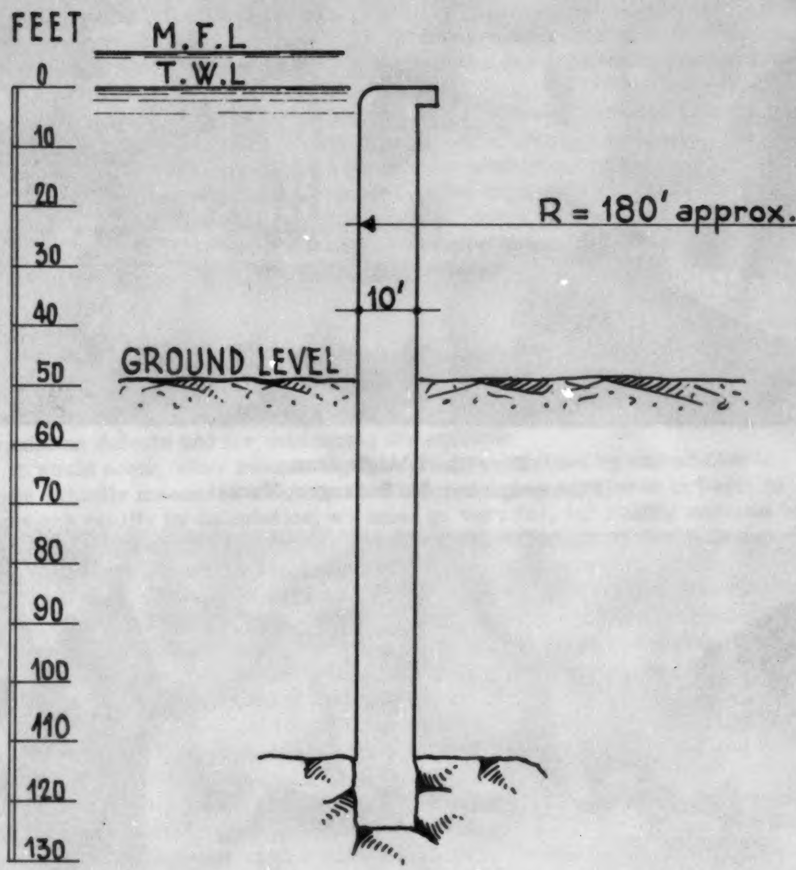

Fig. 11
Bioge Dam
Cross-Section

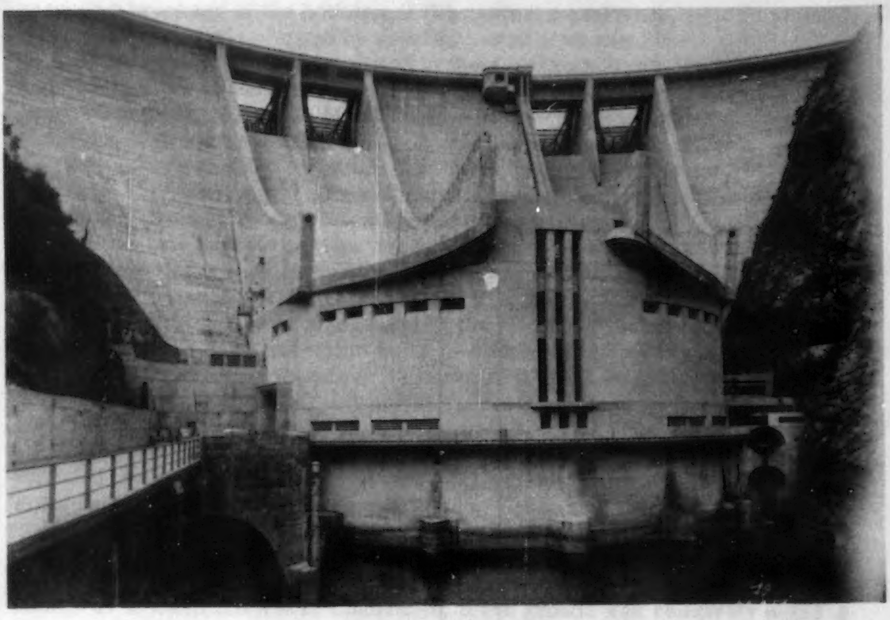

Fig. 12. L'Aigle Dam
(Photograph by: H. Baranger, Paris)

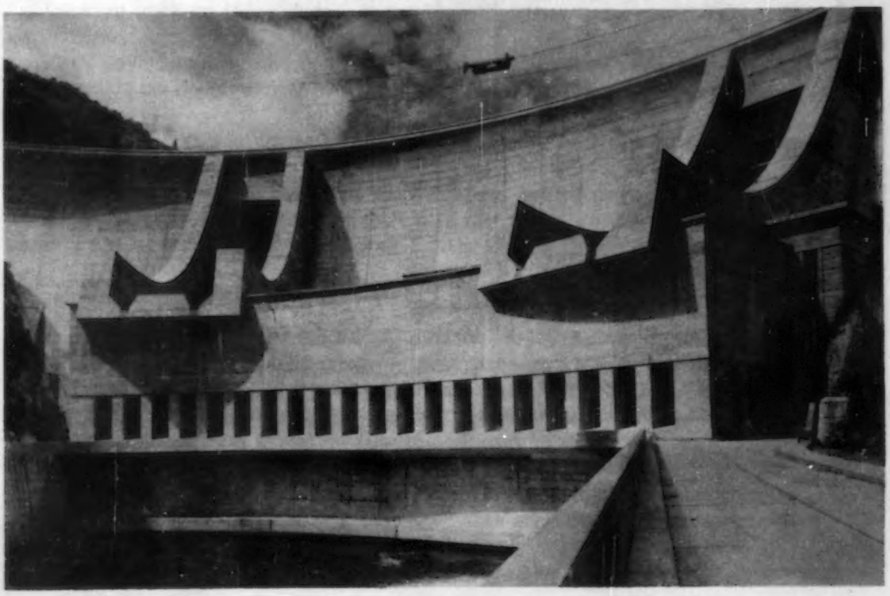

Fig. 13. Chastang Dam
(Photograph by: H. Baranger, Paris)

In any case, experience proves that extension and even cracks in arch dams are not dangerous.

It follows that economy in stresses, particularly at each end of a dam, and economy of materials are far from being always predominant considerations. There are even cases where it is better to keep them to a merely secondary status. Thus in the case of small or average dams, even if they are daring as at Le Gage, it is often preferable to keep to simple shapes, even at the cost of running the risk of fissuration.

On the contrary, extra amounts of volume are accepted as in the United States for instance, where gravity dams win the day from an economic standpoint.

Generally speaking, as far as arches are concerned, striving after economy in volume is most certainly a paying proposition in the case of large-scale dams and those whose shapes are more than usually out of the ordinary.

This is the case of Kariba arch dam on the Zambezi (Fig. 14). Its adoption for a very wide valley will result in great economy as compared to a gravity dam and will above all greatly simplify the question of the diversion of the river.

Calculation

The calculation of arch dams is difficult, owing to the very reasons whereby they resist all the various forces so well: that is to say, their great hyperstatic capacities, which enable them to compensate both for any unexpected foundation defects and for inadequate dimensions.

It would seem, after comparing the stresses obtained by calculation to those actually measured hitherto, that before we can hope even to begin to approach reality by calculation, we must go very far, for reality escapes our grasp; particularly as far as thermal effects are concerned, and it is more or less unpredictable where foundation reactions are concerned.

The United States engineers who systematized and developed so extensively the Trial Load Method had the merit of being the first to take account of the deformability of foundation rock. It is this that has given the method greater accuracy than others. Nevertheless, many French experiences, followed by European ones, have revealed that this deformability could readily be very much greater in the actual foundations as a whole than on a sample of rock or according to tests carried out within a limited area on the damsite and that it depends on the site, the banks and the elevation.

Then, the more daring, the thinner, the more loaded the dam, the greater the importance of local rock defects. The experience of Le Gage shows that these defects can cause such foundation changes that several series of calculations would have had to be carried out, on different support assumptions, so as to cover all the facts.

All this is enough to discourage the toughest Engineer, when we think of the extreme complexity of even the standard Trial Load calculations. In any case, the question as to whether it is all worth while arises, for we have another quicker, more reliable and cheaper method of investigation at our disposal: structural models.

Structural Model

The Italian and Portuguese schools of thought in the matter of small-scale models have got Designers into the habit of thinking that such models provide

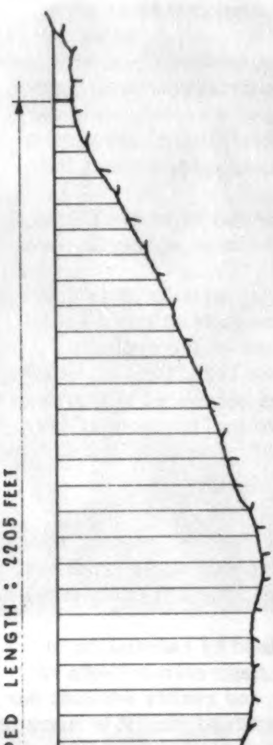

Fig. 14. Kariba Dam
Developed Upstream Elevation

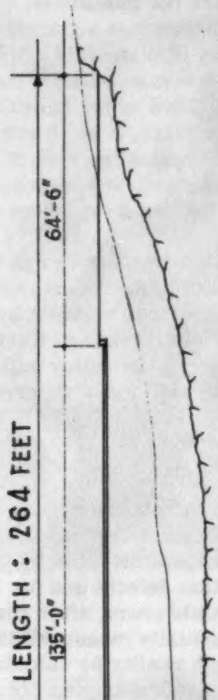

Fig. 15a. Moulin Ribou Dam
Developed Downstream Elevation

sufficient certainty for going boldly ahead, without wasting too much time - even with the help of electronics - on solving equations. Equations in any case have one great failing: if the results actually achieved are not consonant with the results expected, it will be difficult to summon up the courage to start all over again.

So the best thing to do is to put up with simplified calculations, giving results within two limits and to get from them the courage for making up one's mind for choosing a given design. This is then immediately elaborated on a model, but with the clearly defined intention of retouching this model several times over so as to gain little by little on volumes and stresses, as there is only a very slight chance of the first model working out right from the start.

This method has borne its fruit in modern progress.

Present-day applications are very different from those fashionable a few years ago, consisting in using the models simply to confirm the correctness of the method of calculation adopted, and specially that of the Trial Load Method, then a subject of controversy. Today, over-exacting calculations, should they be necessary, particularly for calming the troubled consciences of a few people, are only undertaken as a final check, after all the dimensions have been established in the laboratory.

Future Prospects

It is this method which has enabled us to go further and further in the use of wide valleys - even valleys wide at their bases - without any fear of excessive deflections in the central cantilevers, as in the cases of Pievo de Cadore, Kariba.

There is no doubt at all that it will enable us to go yet further in this direction, though we had hoped to be able to rely on calculations alone for designing a particularly long but low dam, with two hinges: Moulin Ribou (Fig. 15).

In the case of Roselend arch dam (Fig. 16), apart from some rough calculations on plunging arches based on a method I inaugurated about twenty-five years ago, the work was completely based on models. The principles of this structure are now well known and can be summed up as follows:

The arch rises high up above the gorge and it comprises two parts composing a single monolith - the lower part is a standard arch, with the usual abutments on the banks; above this there is an arch without any abutments, but sloping crosswise along a long oblique line. Here the plunging arch effects draw down to the foundations the thrust of the water and the dead weight, as if the abutments were somehow incorporated in the arch itself.

Another factor of progress for arch dams is the excellent quality and performances of modern concrete, such as large aggregate concrete and specially gap-grading concrete, containing up to 60 per cent of 100 to 250 mm (4 in to 10 in) aggregate, the strength of which, at one year old, attains more than 350 kg/cm2 (5,000 lb/in2) at moderate cement contents (220 kgs per cubic meter) (370 lb per cu. yd.). This allows for working stresses exceeding 100 kg/cm2 (1,400 lb/in2) for the usual applications.

The progress already accomplished in multiple arch dams, which share the advantages of pure arches, and the great future opening out before these structures, deserve special mention. Some designers in Europe and Africa, who are up against the problem of saving materials because of transport difficulties and expensive cement, and still having the advantage of relatively cheap labour, still favour this type of dam.

The best example is that of Oued Mellègue (Fig. 17), completed recently. I hope, however, to go one better very shortly.

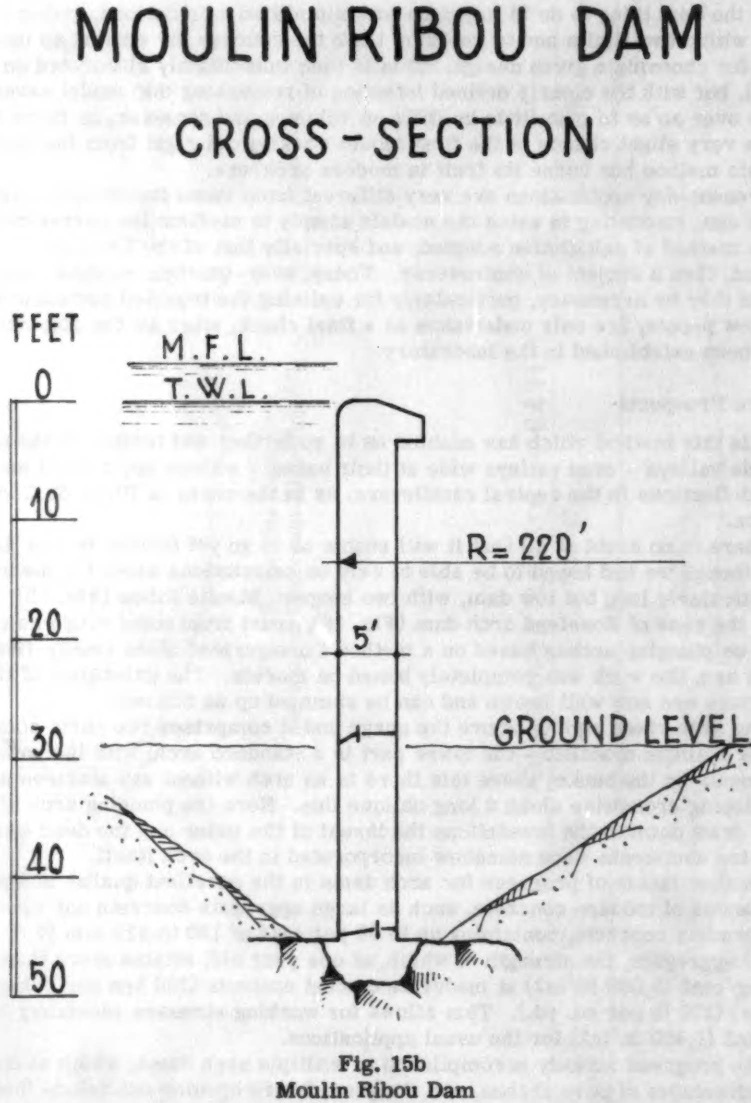

Fig. 15b
Moulin Ribou Dam
Cross-Section

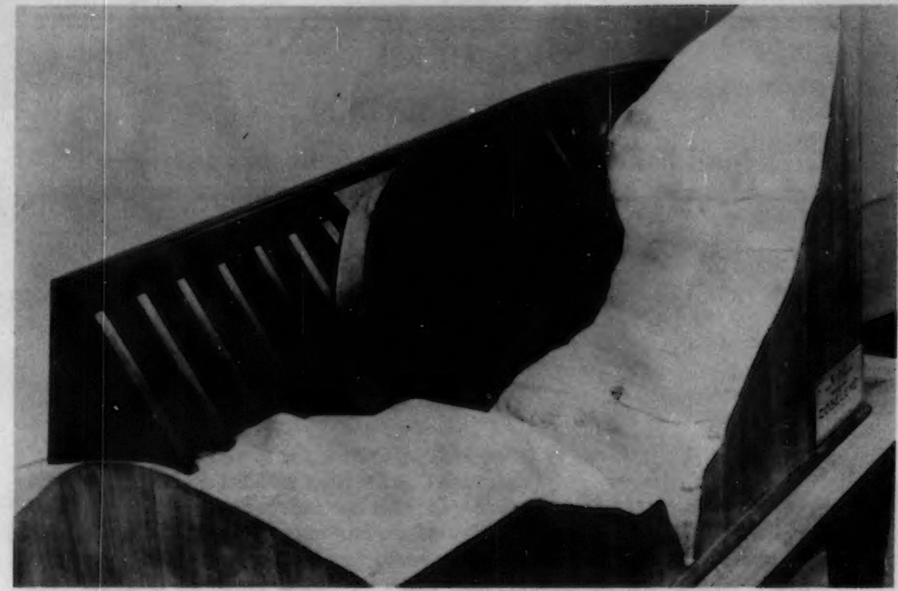

Fig. 16
Model of Roselend Dam

Fig. 17
Oued Mellègue Dam
(Photograph by: J. D. Bossoutrot, Tunis)

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TRIAL LOAD STUDIES FOR HUNGRY HORSE DAM

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(Proc. Paper 960)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June 1956 at Knoxville, Tennessee.

Since the last symposium on masonry dams (April 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, two papers have been approved: "Arch Dams—Their Philosophy", (Proc. Paper 959) by Andre Coyne, Hon. M. ASCE, and "Arch Dams: Trial Load Studies for Hungry Horse Dam," (Proc. Paper 960) by Robert E. Glover, M. ASCE, and Merlin D. Copen.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

A brief history of the development of the Trial Load method for stress analysis of arch dams is first given and the application of these procedures to the design of the Hungry Horse Dam is then described in detail. The Kirchhoff uniqueness theorem of the Theory of Elasticity is used to show that the radial, tangential and twist adjustments of a complete Trial Load analysis are adequate to meet all the requirements for a correct stress analysis of the structure under the loading conditions assumed. However, local stress concentrations may profit from study by photoelastic means and the results of strain gaging may indicate that some factors should be included which are now ignored. Stress distributions as obtained by a radial adjustment only and by

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a complete Trial Load analysis are compared. An analysis based upon the abutment configuration of the dam, as built, is compared to the results of the design analysis based upon the abutment configuration developed from the preliminary investigations at the site to show the influence of such changes upon the maximum stress in the dam.

Development of the Trial Load Method at the Bureau of Reclamation

Before describing the results of the stress computations for the Hungry Horse Dam it may be well to trace briefly, the development and characteristics of the method by which they were obtained.

The U. S. Reclamation Service, which was the predecessor of the present Bureau of Reclamation came into existence on June 17, 1902 when the act creating it became effective. This organization began the design of arch dams by the Trial Load method almost immediately(17) and before the summer of 1905 was passed they had completed the design of two of the worlds great dams by this process. These two dams were the Pathfinder and Shoshone (now Buffalo Bill) dams in Wyoming. Because the canyons in which these two dams were to be built were very similar, one analysis was used for both of them. The analysis was made by the Consulting Engineers George Y. Wisner, M. ASCE and Edgar T. Wheeler, M. ASCE. The "Feature History of the Shoshone Dam"(3) carries the following statement: "No decision as to the type of dam was reached until the summer of 1905. At this time the late George Y. Wisner, Consulting Engineer, acting under recommendation of the Board of Engineers for the Reclamation Service, presented a report embodying a design for high and short masonry dams which was adopted for the Shoshone and Pathfinder sites. The type of structure as adopted was the result of careful study by Mr. Wisner, assisted by Mr. Edgar T. Wheeler, and the subject is covered in a report by Mr. Wisner entitled: 'Investigation of Stresses in High Masonry Dams of Short Spans' which has been published,(2) with diagrams, in the technical journals".

The account of their work(2) shows a very clear comprehension of the structural action in arch dams. They adjusted the arch and crown cantilever deflections only but recognized the need for a complete radial adjustment and computed deflections at points between the crown and abutment to investigate the remaining discrepancies at these points. They accounted for temperature changes as well as the loads produced by water pressure, and were aware of foundation deformations and twist action. Both of these dams were completed and in service by January 16, 1910 and are still in excellent condition. At the time it was completed, the Shoshone Dam was the highest in the world. The next advance was made by the Bureau Engineers Julian Hinds, M. ASCE, C. H. Howell, M. ASCE and A. C. Jaquith who developed procedures for making a complete radial adjustment.(7) The Gibson Dam in Montana was the first to be designed on this basis. Shortly thereafter the senior author added the twist and tangential adjustments and when the model testing work organized by Mr. I. E. Houk, M. ASCE, had produced deflection measurements on the model of Gibson Dam, an analysis was made(9) to determine whether the Trial-Load procedure, as then developed, was adequate. It was found that the twist and tangential adjustments were needed and that when these were added a close agreement between observed and computed displacements was obtained.(9) This result was confirmed by later model testing.(14) In making

these computations, foundation deformations were accounted for by using the formulas developed by Dr. Fredrik Vogt,[5] M. ASCE and by noting the peculiar boundary condition at the top of the dam[1-11] which was brought to our attention by Dr. H. M. Westergaard, M. ASCE.[21] The Owyhee Dam, in Oregon, was the first dam to be designed by the Trial Load procedure which included twist and tangential adjustments. The computation procedures were later improved by R. S. Lieurance who replaced summations with integrals. These were tabulated[12] by Mr. F. D. Kirn using series formulas[11-12] devised for this purpose.[12] The additional water pressures applied to dams during earthquakes[8] were treated by Dr. H. M. Westergaard. The mathematical theory of the conduction of heat in solids was used for preparation charts for temperature changes in the concrete due to external temperature changes[10] and for cooling of concrete dams by embedded pipes.[15] If an arch ring does not meet the abutment along a radial line the arch is considered to terminate at a radial line and the deformations of the triangular portion of concrete lying between this radial line and the actual abutment line are included by using formulas developed by Dr. Westergaard from strain-energy considerations. All of these developments found application in the design of the Hungry Horse Dam.

When the radial, tangential and twist adjustments are made the analysis satisfies the Kirchhoff uniqueness theorem[6] of the Theory of Elasticity as nearly as it is possible to do so with an analysis based upon prismatic elements which extend through the dam from the upstream to the downstream face.[11] Such an analysis may be said to be complete. The Trial Load method shares this type of element with the ordinary slab and thin shell theories.

If the prismatic element described above is further subdivided into approximately cubical elements cut out by surfaces normal to a radius it will be found that continuity conditions in thick dams are not completely satisfied near the upstream and downstream faces even though the adjustments are perfectly made at the midpoint of the prism. In Bureau practice it has been customary to use special means[13] to account for the departures from the linearity of stress distribution postulated for the prismatic element. Photoelastic methods should also be effective for these purposes. The way in which the three adjustments account for the three displacements and the three rotations experienced by the prismatic element as the dam passes from the unstrained to the strained state is shown in the following table:

Table I

Effects of the adjustments

Adjustment	Radial displacement	Tangential displacement	Vertical displacement	Rotation about the radius	Rotation about the tangent	Rotation about the vertical
Radial	*					
Tangential		*	*	*		
Twist					*	*

The Kirchhoff uniqueness theorem not only gives assurance that there can be but one stress system for any given load situation but tells by implication, what conditions must be met if this unique stress system is to be found. These conditions are the following: (1) The condition of equilibrium must be satisfied everywhere. (2) The condition of continuity must be satisfied throughout the structure and (3) the boundary conditions must be satisfied. The Trial Load method meets these requirements through the three adjustments described in Table I. This method is therefore complete in the sense that no other adjustments are required. The purpose of a complete Trial Load analysis is to find that distribution of stress which must be present in the dam under the assumed conditions of use. It is not a short cut method in any sense and its use will require an expenditure of considerable time and effort. However, it has advantages as a design tool for large arch dams where sound economy, and the safety of lives and investments are paramount.

If these purposes are to be attained, care must be taken to see that the actual conditions are imposed upon the analysis. It is especially necessary to scrutinize all stereotyped assumptions and to discard them if they do not accord with the facts. The shortcomings of one of these were disclosed by the strain-meter data obtained from the Shasta Dam.(18) These measurements showed conclusively that the stresses are strongly influenced by the construction procedure in spite of the fact that accepted design methods generally ignored these factors. It will be noted in the descriptions which follow that a careful accounting was made in the Hungry Horse Trial Load studies for effects of the construction procedure. Of particular importance were the load temperature and structural conditions at the times of grouting. Cooling by embedded pipes was used to bring the concrete temperature to a predetermined level before the grouting in each stage was done.

It may be well to point out that strain gage data from dams in service will not generally conform to the results of a study such as has been described. The reason for this is that the designer generally chooses a maximum possible set of loadings as a basis for his design whereas the service conditions will generally be less severe. In the present case maximum water loadings, temperature, earthquake and ice loadings were used as a design basis but, in service, water levels will generally be less than maximum, ice may not be present and earthquakes will seldom occur. It should not be surprising, therefore, if the strain gage data indicate stresses differing from those obtained from the Trial Load analyses. Another factor which influences the stresses in dams is temperature change. The temperature changes associated with the march of the seasons affect the temperatures of the concrete throughout the thinner dams and penetrate to depths of about 50 feet in the thicker dams. The daily changes and the erratic fluctuations caused by changes of the weather are important near the surface but do not penetrate as deeply as the yearly changes. Potentially, these changes are capable of producing stresses comparable to those due to the water load. In the present design allowances were made for changes in the mean temperature of the arch rings but no accounting was made for variable temperature distributions through the arch rings. It is possible that the results of strain-gagings will show that this factor is of sufficient importance so that some accounting should be included for it in the stress analysis.

Temperature Control

In order to obtain the most effective structural action in an arch dam it is desirable to do the grouting, which makes it behave as a structural entity, at a time when the mean temperature is at its lowest point in the yearly cycle of temperature change.[10] If the dam is so large that it will not come to thermal stability with its surroundings during the construction period then an embedded pipe cooling system may be used to bring the concrete temperatures under control.[15] At the Hungry Horse site river water was used for cooling and the temperatures were brought down somewhat below the estimated final configuration to improve structural action in the dam. If the grouting is done about the first of March the condition of minimum mean temperature is usually obtained since the mean concrete temperatures lag behind the external changes by about one-eighth of a year.[10]

In those cases where winter stops the placement of concrete the exposed top surfaces of the blocks are subjected to severe temperature stresses which have a tendency to split the top of the block. If this happens the crack so formed has a tendency to propogate up through the new concrete when placement is resumed.

An innovation was tried at the Hungry Horse Dam by insulating the tops of some of the blocks with planer shavings.[16] This proved to be very effective and the concrete temperatures in the blocks so treated did not reach the freezing point during the winter. The use of thermal insulation for maintaining favorable curing conditions in concrete placed in cold weather has since proved its effectiveness on other jobs.[20]

Design Study

The Hungry Horse Dam has been constructed on the South Fork of the Flathead river, the site for the dam being about nine miles southeast of Columbia Falls, Montana. Construction was completed in 1952. The appearance of the completed dam is shown in figure 1. Figure 2 shows a general plan of the dam and a profile along the arch center-lines. Cross sections of the Cantilever elements used in the design studies are shown on figure 3.

Estimates of the thermal properties, the density and the structural and elastic properties of the concrete to be used in the dam were obtained from the Bureau Laboratories. A special study was made to determine the maximum ice thrust which might be expected to be exerted on the dam. The estimated schedule for placing concrete, temperature variations in the concrete and times of grouting the contraction joints were furnished by the Temperature Control group in the Dams Division.

The design of the dam has a uniform thickness of 35 feet along the top arch ring, a uniform thickness of 55.86 feet at elevation 3500, and a uniform thickness of 81.0 feet at elevation 3450. Below elevation 3448.5 the arch rings are variable in thickness from their crowns to the abutments. The length of the dam along its crest was estimated to be 2,060 feet corresponding to a half central angle of the top arch ring of 50 degrees. The dam, as analyzed, is 321 feet thick at the base of the crown cantilever and its maximum height was assumed to be 515 feet. Since these studies were made, however, excavations at the base of the dam have been completed and the official height of the structure has been determined to be 564 feet. This is the distance from the top of the dam to the lowest point in the foundation for

which mass concrete prices apply. The effect of these changes was evaluated by a supplemental Trial Load study, to be described later, which also accounted for the actual construction procedure.

Basic Design Data Used in Analyses

Trial Load stress studies made for the proposed design of Hungry Horse Dam were based on the following design data:

a. Crest of dam, elevation 3565.
b. Base of crown cantilever, elevation 3050.
c. Normal high water surface, elevation 3560.
d. Reservoir water surface during times of maximum drawdown, elevation 3250.
e. Increase in horizontal pressure due to silt accumulations, if any, were not included in the analyses.
f. Effects of tailwater were neglected.
g. Ice pressure, 5 tons per linear foot.
h. Thickness of ice sheet, 2.25 feet.
i. Maximum horizontal earthquake assumed to have an acceleration of one-tenth of gravity, a period of vibration of one second, and a direction of vibration normal to the axis of the dam at the line of centers.
j. Effects of vertical earthquake were not included in the analyses.
k. Sustained modulus of elasticity of concrete in tension and compression, 3,000,000 lb/in^2.
l. Sustained modulus of elasticity of foundation and abutment rock 3,000,000 lb/in^2.
m. Modulus of elasticity of concrete in shear, 1,250,000 lb/in^2 reduced to 1,000,000 lb/in^2 in calculating the detrusions caused by radial shears to allow for the nonlinear distribution of shearing stresses between the upstream and downstream faces of the dam.
n. Poisson's ratio for concrete, 0.20.
o. Poisson's ratio for foundation and abutment rock, 0.20.
p. Unit weight of concrete, 150 pounds per cubic foot.
q. Unit weight of water, 62.5 pounds per cubic foot.
r. Coefficient of thermal expansion of concrete 0.000,005,97 feet per foot per degree Fahrenheit.

Basic Assumptions Used in Analyses

The following basic assumptions were used in the Trial Load analyses:

a. For purpose of making the analyses, the arch rings above elevation 3400 were assumed to have radial abutments and below this elevation they were assumed to have triangular abutments.
b. Arch rings above elevation 3400 are symmetrical about their crowns and are nonsymmetrically loaded. Below elevation 3400, the arch rings are nonsymmetrical about their crowns and are nonsymmetrically loaded.
c. Foundation and abutment rock formations at the site of Hungry Horse Dam have adequate strength to safely carry the loads transmitted by the dam.
d. The concrete in the dam will be homogeneous, uniformly elastic in all directions, and strong enough to carry the applied loads with stresses well below the elastic limit.

e. The dam will be thoroughly keyed into the foundation and abutment rock throughout its contact with the canyon profile so that arches may be considered as fixed with relation to the abutments, and cantilevers as fixed with relation to the foundation.

f. Contraction joints in the dam will be thoroughly grouted according to the grouting schedule and it is assumed that these joints will remain grouted during the life of the structure.

Loading Conditions and Studies Made

Stresses determined from the Trial Load studies include effects due to the construction and grouting programs, horizontal earthquake, ice pressure and cooling of the concrete prior to grouting the contraction joints. Assumptions, loading conditions, and analyses made to include these effects were as follows:

A. Stage 1

(a) Dam built to elevation 3300.
(b) Contraction joints from foundation to elevation 3300 grouted when the reservoir is empty.
(c) Analysis made: Gravity analysis, dead load carried by cantilevers.

B. Stage 2

(a) Crest of dam raised from elevation 3300 to elevation 3440.
(b) Reservoir water surface raised to elevation 3325.
(c) Contraction joints above elevation 3300 ungrouted.
(d) All loads carried by cantilever action above elevation 3300 and by arch and cantilever action below elevation 3300.
(e) Analysis made: Complete Trial Load analysis for portion of dam below elevation 3300. This analysis includes effects due to tangential shear and twist, the reservoir water load to elevation 3325, and the weight of the concrete above elevation 3300. Temperatures used in the study are shown in column 1 of Table 2.

C. After the concrete in the lifts between elevations 3300 and 3400 has been cooled to the required closure temperatures it was assumed that contraction joints in this portion of the dam would be grouted when the reservoir water surface is at elevation 3325.

D. Stage 3

(a) Crest of dam raised from elevation 3440 to elevation 3565.
(b) Reservoir water surface raised from elevation 3325 to elevation 3425.
(c) Contraction joints above elevation 3400 ungrouted.
(d) All loads carried by cantilever action above elevation 3400 and by arch and cantilever action below elevation 3400.
(e) Analysis made: Complete Trial Load analysis for portion of dam below elevation 3400. This analysis includes effects due to tangential shear and twist; raising the reservoir water surface from elevation 3325 to elevation 3425; and the weight of the concrete above elevation 3440. Concrete temperatures used in the analysis are listed in column 2 of Table 2.

E. After the concrete in the lifts between elevations 3400 and 3565 has been cooled to the required closure temperatures it was assumed that contraction joints in this portion of the dam would be grouted when the reservoir water surface is at elevation 3425.

F. Stage 4 Completed Dam

(a) Arch and cantilever action occurs in entire dam.
(b) Reservoir water surface raised from elevation 3425 to elevation 3560.
(c) Analysis made: Complete Trial Load analysis of entire dam. The analysis includes effects due to tangential shear and twist, raising the reservoir water surface from elevation 3425 to elevation 3560, horizontal earthquake and ice pressure. Concrete temperatures used in the analysis are shown in column 3 of Table 2.

G. Stresses in the completed dam were obtained by superimposing stresses calculated from the analyses listed under stages 1, 2, and 3 on the stresses determined from the analysis for stage 4.

Table 2

EFFECTIVE CONCRETE TEMPERATURES
USED IN DESIGN STUDY

	Stage 2	Stage 3	Stage 4
Elevation	Temp. ° F.	Temp. ° F.	Temp. ° F.
3565			-1.4
3500			+4.3
3450			+6.0
3400		+3.3	+2.7
3350		+3.0	+2.6
3300	+4.3	+2.4	-1.4
3250	+3.1	+2.4	-0.5
3200	+3.0	+2.0	0
3150	+3.7	+2.4	-1.1

Plus sign means temperature rise.
Minus sign means temperature drop.
Temperatures in Table 2 reflect the effects of subcooling the concrete to 38° F. prior to closure of contraction joints.

Autogenous Shrinkage

Specifications for Hungry Horse Dam required that a predetermined per cent of fly ash, or a pozzolan of similar characteristics, by weight of Portland cement be used to make the concrete for the dam.

Laboratory tests, of concrete specimens containing fly ash and cement, indicated that such concrete may be subjected to autogenous shrinkage.

Autogenous shrinkage occurring in an arch dam after the joints are grouted will affect stresses in the dam in the same manner as if a temperature drop occurs in the concrete. However, in a report to the Chief Engineer by the

Board of Consultants on "The Use of Portland Pozzolan Materials in Hungry Horse Dam" dated July 29, 1948, it is stated on page 4, second paragraph that:

"Pending the proof that such autogenous shrinkage will occur with the combination of materials which are finally selected, it is recommended that any effects of autogenous shrinkage be neglected in the design calculations".

Effects of autogenous shrinkage were therefore omitted from the analyses discussed herein.

Later results(19) from the tests did indicate some autogenous shrinkage but, although there is considerable scatter among these results, it appears that this shrinkage generally came early enough so that substantial stability was reached at the times when grouting was done. The recommendation, therefore, seems to have been justified.

Radial Trial Load Analysis

The results of a Trial Load adjustment of the radial deflections only is included for purpose of comparison. Such a comparison is useful since it shows the effects of the tangential shear and twist adjustments and other refinements of the analysis on the computed stresses in the dam. The effects of construction procedures and the grouting program were not included in this analysis. This study was based upon the following conditions:

a. Reservoir water surface elevation 3560.
b. Ice sheet assumed to exert a horizontal pressure of 5 tons per linear foot at elevation 3558.75.
c. Effects of tailwater and uplift are not considered.
d. Temperature of concrete at time of grouting contraction joints assumed to be 38^{o} Fahrenheit.
e. Temperatures used in analysis are minimum stable temperatures modified by effects of sub-cooling.
f. Earthquake assumption- Dam moves upstream and downstream horizontally in the direction of the line of centers. Increased water pressure acts equally on all cantilevers. Period of vibration 1.0 second. Acceleration 0.1 gravity. Effects of vertical acceleration are not included.
g. Modulus of elasticity of concrete and abutment rock 3,000,000 pounds per square inch.
h. Poisson's ratio of concrete and abutments; 0.2.
i. Unit weight of concrete 150 pounds per cubic foot.
j. Coefficient of thermal expansion of concrete 0.000,005,97 feet per foot per degree Fahrenheit.

The stresses obtained from this study and their modification by the twist and tangential adjustments and other refinements of the complete design study are included in Tables 3 and 4. The effects of Poisson's ratio were not included in the adjustments of either of these studies.

The stresses in Table 3 act in the horizontal plane of the arch and those in Table 4 act in the vertical plane of the cantilever. However, in both cases, when the surface is inclined to the radial or horizontal planes, on which the stresses were originally computed, a factor was applied to obtain the maximum stress at the surface. This factor is the reciprocal of the square of the cosine of the angle by which the surface departs from the normal to the plane on which the stress was originally computed. Although a comparison of the values in Tables 3 and 4 will show that these direct stresses were generally

decreased by the twist and tangential adjustments a further comparison should be made. Since the twist and tangential adjustments bring shearing stresses on horizontal and vertical planes into the analysis it becomes possible to compute principal stresses and the maximum principal stress, so obtained is sometimes greater than either of the direct stresses given in the Tables 3 and 4. The two greatest principal stresses obtained from the analysis which includes the tangential and twist adjustments are on the downstream face. On the left abutment a stress of 731 pounds per square inch occurs at elevation 3200. On the right abutment a stress of 724 pounds per square inch occurs at elevation 3250. The highest stress obtained from the radial adjustment analysis is 571 pounds per square inch acting horizontally at elevation 3400 on the right abutment. In this case, therefore, while the stresses on horizontal and vertical planes were generally decreased by the tangential and twist adjustments the maximum computed stress was increased. This increase comes from the shear stresses introduced by the tangential and twist action. The maximum stress obtained from the complete Trial Load study is about 22 per cent higher than the maximum stress obtained from the radial adjustment only.

The maximum compressive principal stress at the upstream face, obtained from the complete analysis, is 299 pounds per square inch and occurs at the right abutment at elevation 3150. Small tensile principal stresses of 30 pounds per square inch or less occur at elevations 3300, 3350, 3400 and 3450 at the right abutment and at elevations 3150 and 3450 at the left abutment.

A compressive stress of 750 pounds per square inch was the maximum to be permitted in the design.

Final Trial Load Study

Since a substantial increase in excavation at the abutments of the arches was necessary over that assumed in previous studies, and there were variations in the construction program from that assumed, it was considered essential that a study be made of the stress conditions in Hungry Horse Dam as it was constructed.

A complete Trial Load analysis of the dam was made, including the construction and grouting program, effects of nonsymmetrical arches and triangular abutments where applicable. Data on concrete placing, grouting and temperature changes were obtained from the Temperature Control Unit. Thermal coefficient, modulus of elasticity and unit weight of concrete, modulus of elasticity of foundation and abutment rock were provided by the laboratory from actual test data. Data pertinent to the final study are shown in Table 5.

A plan of the dam as it was analyzed, the maximum section and a developed profile are shown on figure 2.

Basic Design Data Used in Analysis

a. Crest of dam, elevation 3565.
b. Base of crown cantilever, elevation 3050.
c. Normal reservoir water surface, elevation 3560.
d. Increase in horizontal pressure due to silt accumulations if any, were not included in the analysis.
e. Effects of tailwater were not included.

TABLE 3

EFFECTS OF TANGENTIAL SHEAR AND TWIST
ON ARCH STRESSES AT CROWN AND ABUTMENT SECTIONS
EFFECTS OF POISSON'S RATIO NOT INCLUDED

Elev.		Stresses are in pounds per square inch								
		Left Abutment			Crown			Right Abutment		
		Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist
3565	Ext.	+ 300	+ 89	- 211	+ 340	+ 296	- 44	+ 301	+ 125	- 176
	Int.	+ 167	+ 122	- 45	+ 126	+ 172	+ 46	+ 165	+ 90	- 75
3500	Ext.	+ 242	+ 102	- 140	+ 440	+ 338	- 102	+ 210	+ 107	- 103
	Int.	+ 358	+ 202	- 156	+ 157	+ 257	+ 100	+ 391	+ 323	- 68
3450	Ext.	+ 170	+ 130	- 40	+ 479	+ 368	- 111	+ 114	+ 135	+ 21
	Int.	+ 456	+ 245	- 211	+ 140	+ 221	+ 81	+ 513	+ 323	- 190
3400	Ext.	+ 143	+ 188	+ 45	+ 510	+ 374	- 136	+ 4	+ 96	+ 92
	Int.	+ 411	+ 245	- 166	+ 89	+ 155	+ 66	+ 571	+ 405	- 166
3350	Ext.	+ 99	+ 189	+ 90	+ 518	+ 366	- 152	- 80	+ 104	+ 184
	Int.	+ 385	+ 307	- 78	+ 20	+ 104	+ 84	+ 557	+ 375	- 182
3300	Ext.	- 5	+ 163	+ 168	+ 500	+ 329	- 171	- 87	+ 96	+ 183
	Int.	+ 455	+ 335	- 120	- 29	+ 34	+ 63	+ 534	+ 406	- 188
3250	Ext.	- 69	+ 121	+ 190	+ 478	+ 302	- 176	- 114	+ 121	+ 235
	Int.	+ 479	+ 345	- 134	- 74	+ 10	+ 84	+ 521	+ 402	- 119
3200	Ext.	- 67	+ 122	+ 189	+ 393	+ 267	- 126	- 67	+ 135	+ 202
	Int.	+ 406	+ 311	- 95	- 71	- 12	+ 59	+ 403	+ 348	- 55
3150	Ext.	- 32	+ 114	+ 146	+ 291	+ 228	- 63	- 38	+ 130	+ 168
	Int.	+ 256	+ 200	- 56	- 76	- 38	+ 38	+ 268	+ 268	0

Ext. means stress at extrados of arch.
Int. means stress at intrados of arch.
Tension stresses are indicated by minus sign in table.

TABLE 4

EFFECTS OF TANGENTIAL SHEAR AND TWIST ON STRESSES IN CROWN CANTILEVER AND CANTILEVERS D AND E EFFECTS OF POISSON'S RATIO NOT INCLUDED

Elev.		LOADING CONDITIONS-B Stresses are in pounds per square inch								
		Cantilever - E			Crown Cantilever			Cantilever - D		
		Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist
3565	U.	0	0	-	0	0	-	0	0	-
	D.	0	0	-	0	0	-	0	0	-
3500	U.	+ 53	+ 58	+ 5	+ 57	+ 61	+ 4	+ 52	+ 51	- 1
	D.	+ 64	+ 59	- 5	+ 60	+ 56	- 4	+ 66	+ 68	+ 2
3450	U.	+ 63	+ 72	+ 9	+ 71	+ 76	+ 5	+ 62	+ 67	+ 5
	D.	+ 136	+ 129	- 7	+ 125	+ 118	- 7	+ 137	+ 130	- 7
3400	U.	+ 66	+ 80	+ 14	+ 79	+ 86	+ 7	+ 69	+ 79	+ 10
	D.	+ 203	+ 184	- 19	+ 180	+ 170	- 10	+ 202	+ 189	- 13
3350	U.	+ 67	+ 86	+ 19	+ 86	+ 97	+ 11	+ 77	+ 93	+ 16
	D.	+ 275	+ 250	- 25	+ 243	+ 226	- 17	+ 264	+ 246	- 18
3300	U.	+ 69	+ 94	+ 25	+ 93	+ 112	+ 19	+ 88	+ 112	+ 24
	D.	+ 350	+ 318	- 32	+ 305	+ 279	- 26	+ 330	+ 304	- 26
3250	U.	+ 72	+ 103	+ 31	+ 105	+ 129	+ 24	+ 102	+ 136	+ 34
	D.	+ 432	+ 396	- 36	+ 365	+ 330	- 35	+ 398	+ 366	- 32
3200	U.	+ 82	+ 118	+ 36	+ 122	+ 153	+ 31	+ 124	+ 164	+ 40
	D.	+ 512	+ 476	- 36	+ 418	+ 373	- 45	+ 466	+ 438	- 28
3150	U.	+ 95	+ 137	+ 42	+ 146	+ 183	+ 37	+ 152	+ 196	+ 44
	D	+ 578	+ 545	- 33	+ 463	+ 408	- 55	+ 518	+ 503	- 15
3050	U.				+ 204	+ 255	+ 51			
	D.				+ 542	+ 463	- 79			

U. means stress at upstream face of cantilevers.
D. means stress at downstream face of cantilevers.
Tension stresses are indicated by minus sign in table.

f. Ice pressure, 5 tons per linear foot with a sheet of ice 2.25 feet thick.
g. Maximum horizontal earthquake has an acceleration of one-tenth gravity, a period of vibration of one second, and a direction of vibration parallel to the line of centers.
h. Effects of vertical earthquake acceleration were not included.
i. Sustained modulus of elasticity of concrete in tension and compression, 3,940,000 pounds per square inch.
j. Sustained modulus of elasticity of foundation and abutment rock 4,400,000 pounds per square inch.
k. Modulus of elasticity of concrete in shear, 1,641,667 pounds per square inch reduced to 1,313,333 pounds per square inch in computing the detrusions caused by radial shears to allow for the nonlinear distribution of shearing stresses between the upstream and downstream faces of the dam.
l. Poisson's ratio for concrete and abutment rock, 0.20.
m. Unit weight of concrete, 150 pounds per cubic foot.
n. Unit weight of water, 62.5 pounds per cubic foot.
o. Coefficient of thermal expansion of concrete, 0,000,005,3 feet per foot per degree Fahrenheit.

Basic Assumptions Used in Analysis

a. All arch rings were assumed to have radial abutments except those below elevation 3400 which were assumed to have triangular abutments on the right side.
b. Arch rings above elevation 3400 are symmetrical about their crowns and are nonsymmetrically loaded. Below elevation 3400, the arch rings are nonsymmetrical about their crowns and nonsymmetrically loaded.
c. Foundation and abutment rock formations at the site of Hungry Horse Dam have adequate strength to safely carry the loads transmitted by the dam.
d. The concrete in the dam is homogeneous, uniformly elastic in all directions, and strong enough to carry the applied loads with stresses well below the elastic limit.
e. The dam is thoroughly keyed into the foundation and abutment rock throughout its contact with the canyon profile so that arches may be considered as fixed with relation to the abutment, and cantilevers as fixed with relation to the foundation.
f. Contraction joints were thoroughly grouted according to the grouting schedule and it is assumed that these joints will remain grouted throughout the life of the structure.

Loading Conditions

In determining the stresses from the Trial Load analysis, every effort was made to duplicate as nearly as possible the actual conditions at the damsite and the construction and grouting program followed as the dam was built. The final excavation, effects of earthquake, ice pressure and concrete cooling were included in the study. The loading conditions and the analyses made to include these effects are as follows:

Table 5
Pertinent data relating to the Hungry Horse Dam

Elevation - - feet	Water pressure - lb/ft^2	Pressure due to earthquake* lb/ft^2	Date grouting completed	Average Temperature at closure °F	Average elevation of concrete at closure	Water surface elevation at closure	Range of concrete Temperature** °F Max	Min
3565	0	0	4-17-53	38.0	3565	3376	59.5	33.5
3500	3750	811	4- 3-53	37.6	3565	3373	53.5	41.0
3450	6875	1178	3-30-53	37.6	3565	3371	50.5	43.0
3400	10000	1472	5- 9-52	38.4	3446.5	3292	48.1	43.7
3350	13125	1713	5- 4-52	38.6	3438.75	3285	47.3	43.9
3300	16250	1911	4-26-52	37.3	3430.5	3246	46.8	44.0
3250	19375	2069	4-18-52	37.7	3421.25	3208	46.4	44.1
3200	22500	2190	4-13-51	38.7	3233.5	None	46.2	44.2
3150	25625	2275	4- 5-51	38.4	3228.5	None	46.0	44.2
3050	31875	2343						

* Computed by procedures of U.S.B.R. Engineering Monograph No. 11.

** Range of mean temperatures of concrete at the elevations stated. The effects of solar radiation have been included in these estimates.

a. Stage 1

(1) Dam built to elevation 3225.
(2) Reservoir empty, joints ungrouted.
(3) Analysis made: Gravity analysis, dead load carried by cantilevers.

b. Stage 2

(1) Top of dam raised from elevation 3225 to elevation 3450.
(2) Reservoir water surface at elevation 3250.
(3) Dam grouted to elevation 3200.
(4) Analysis made: Radial Trial Load analysis below elevation 3200, including water load to elevation 3250 and the weight of concrete above elevation 3225. Temperatures used are shown in Table 6.

c. Stage 3

(1) Concrete raised from elevation 3450 to elevation 3565.
(2) Reservoir water surface raised from elevation 3250 to elevation 3375.
(3) Dam grouted to elevation 3400.
(4) Analysis made: Radial Trial Load analysis below elevation 3400, including raising reservoir water surface from elevation 3250 to elevation 3375 and the weight of concrete above elevation 3400. Temperature changes used in this analysis are shown in Table 6.

d. Stage 4

(1) Dam grouted to elevation 3565.
(2) Reservoir water raised from elevation 3375 to elevation 3560.
(3) Analysis made: Complete Trial Load analysis of entire dam, including tangential shear and twist effects for Stages 2, 3, and 4, raising reservoir water surface from elevation 3375 to elevation 3560, horizontal earthquake and ice pressure. Temperatures used in this analysis are shown in Table 6.

Table 6

EFFECTIVE CONCRETE TEMPERATURES
USED IN FINAL TRIAL LOAD STUDY

	Stage 2	Stage 3	Stage 4	Final
Elevation	Temp. °F	Temp. °F	Temp. °F	Temp. °F
3565			-4.5	33.5
3500			+3.4	41.0
3450			+5.4	43.0
3400		+3.6	+1.7	43.7
3350		+6.4	-1.1	43.9
3300		+6.7	0	44.0
3250		+6.3	+0.1	44.1
3200	+3.3	+2.0	+0.2	44.2
3150	+4.6	+2.0	-0.8	44.2

Plus sign means temperature rise.
Minus sign means temperature drop.
Temperatures in Table 6 reflect the effects of subcooling the concrete prior to closure of contraction joints.

e. Stresses in the completed dam were obtained by combining the forces and moments from all four stages and computing the desired stresses therefrom.

Comparison of Stresses in Dam As Constructed With Proposed Design

Comparison of Arch Stresses

In Table 7 is shown a comparison of arch stresses in Hungry Horse Dam, as it was constructed, with the proposed design study. The table is self explanatory. It should be noted that the dam as constructed has a maximum arch compressive stress of 535 pounds per square inch as compared to 405 pounds per square inch in the design study. Small tensile stresses found at the crown intrados of the design study are not present in the dam as constructed. Tensile stresses are found at the abutments of the top arch where compression occurred in the design study; this being due probably to a greatly increased temperature drop in the top portion of the dam. Variations will be noted in the arch stresses throughout the dam but, other than those noted, are not of great importance.

Comparison of Cantilever Stresses

Table 8 shows a comparison of cantilever stresses in Hungry Horse Dam, as it was constructed, and as estimated in the proposed design study. The table indicates that, in general, the stresses at the upstream face are higher in the dam as constructed than in the proposed design, while at the downstream face they are lower. The maximum cantilever stress in the dam decreased from 545 pounds per square inch to 521 pounds per square inch in the completed dam. All of the cantilever stress changes were small and of minor importance.

Comparison of Principal Stresses

A comparison of principal stresses for the dam as constructed with the proposed design is shown in Table 9. Orientations of the principal stresses are shown on figure 5. The maximum compressive principal stress is 631 pounds per square inch in the dam as constructed, compared with 731 pounds per square inch in the proposed design. The maximum tensile principal stress in the dam as constructed is 79 pounds per square inch compared with 30 pounds per square inch in the proposed design.

SUMMARY

The amount of excavation required in the lower part of Hungry Horse damsite was more extensive than was originally anticipated. This resulted in a change of arch and cantilever properties which in turn caused a redistribution of loads throughout the dam. The construction and grouting program as well as the depth of water in the reservoir at the time of grouting varied from the original assumptions, resulting in changes in load distribution. The results of these changes on the stresses are shown in the tables included herein.

An over-all appraisal indicates an increase in maximum principal tensile stresses at the right abutment extrados in the lower part of the dam, and a

TABLE 7

COMPARISON OF ARCH STRESSES

AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		Left abut.		3/4		1/2		1/4		Crown		1/4		1/2		3/4		Right abut.	
		Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A
3565	Ext.	-72	+89	+17	+52	+111	+160	+205	+209	+277	+296	+279	+246	+217	+175	+77	+61	-44	+125
	Int.	-43	+122	+50	+57	+147	+159	+188	+240	+197	+172	+200	+204	+150	+150	+175	+123	-40	+90
3500	Ext.	+41	+102	+170	+174	+252	+246	+314	+291	+377	+338	+351	+326	+315	+283	+200	+205	+114	+107
	Int.	+157	+202	+223	+218	+273	+232	+285	+273	+243	+257	+253	+243	+243	+213	+291	+240	+199	+323
3450	Ext.	+105	+130	+177	+196	+242	+234	+326	+315	+393	+368	+380	+353	+306	+301	+229	+216	+141	+135
	Int.	+212	+245	+253	+237	+279	+274	+262	+255	+220	+221	+213	+220	+234	+232	+282	+278	+421	+323
3400	Ext.	+143	+188	+152	+196	+228	+224	+328	+306	+374	+374	+357	+351	+300	+295	+261	+209	+16	+96
	Int.	+233	+245	+271	+272	+248	+278	+192	+217	+161	+155	+219	+177	+207	+222	+242	+294	+535	+405
3350	Ext.	+139	+189	+157	+164	+223	+207	+317	+304	+370	+366	+340	+334	+272	+263	+187	+181	+94	+104
	Int.	+293	+307	+287	+280	+243	+248	+170	+165	+122	+104	+142	+120	+179	+191	+251	+260	+380	+375
3300	Ext.	+137	+163	+177	+155	+246	+196	+328	+277	+373	+329	+344	+307	+272	+246	+196	+186	+128	+96
	Int.	+329	+335	+301	+275	+239	+212	+160	+101	+113	+34	+134	+65	+188	+160	+254	+243	+336	+406
3250	Ext.	+133	+121	+174	+144	+240	+201	+305	+268	+330	+302	+299	+285	+246	+237	+189	+187	+110	+121
	Int.	+329	+345	+272	+223	+192	+141	+118	+55	+88	+10	+115	+33	+169	+108	+218	+198	+265	+402
3200	Ext.	+142	+122			+214	+201			+264	+267			+209	+223			+134	+135
	Int.	+283	+311			+131	+81			+56	-12			+126	+69			+264	+348
3150	Ext.	+136	+114			+209	+192			+248	+228			+218	+210			+163	+130
	Int.	+214	+200			+100	+15			+52	-38			+97	+12			+203	+268

Stresses are in pounds per square inch.
(+) Indicates compression. (-) Indicates tension.
Ext. = extrados of arch.
Int. = intrados of arch.

TABLE 8

COMPARISON OF CANTILEVER STRESSES
AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		H		G		F		E		Crown		D		C		B		A	
		Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A
3565	U	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3500	U	+30	+33	+43	+50	+52	+50	+58	+58	+63	+61	+64	+51	+56	+49	+50	+40	+41	+30
	D	+93	+89	+77	+69	+66	+68	+59	+59	+53	+56	+52	+68	+61	+70	+68	+81	+79	+92
3450	U	+31	+30	+54	+60	+68	+63	+76	+72	+85	+76	+85	+67	+74	+66	+64	+48	+49	+29
	D	+173	+181	+149	+140	+128	+136	+116	+123	+103	+118	+104	+130	+120	+132	+135	+157	+150	+183
3400	U			+77	+81	+83	+73	+88	+80	+100	+86	+96	+79	+97	+82	+95	+68		
	D			+235	+226	+199	+209	+176	+184	+151	+170	+165	+189	+175	+195	+190	+235		
3350	U			+104	+107	+99	+83	+99	+86	+115	+97	+105	+93	+128	+101	+138	+97		
	D			+281	+274	+262	+281	+236	+250	+201	+226	+229	+246	+211	+254	+203	+284		
3300	U					+120	+101	+114	+94	+134	+112	+123	+112	+160	+128				
	D					+335	+357	+301	+318	+248	+279	+291	+304	+265	+314				
3250	U					+148	+121	+132	+103	+156	+129	+149	+136	+196	+162				
	D					+416	+458	+370	+396	+291	+330	+354	+366	+332	+390				
3200	U							+155	+118	+180	+153	+177	+164						
	D							+448	+476	+433	+373	+427	+438						
3150	U							+180	+137	+201	+183	+203	+196						
	D							+521	+545	+382	+408	+500	+503						
3050	U									+242	+255								
	D									+483	+463								

Stresses are in pounds per square inch.
(+) Indicates compression.
U = Upstream edge of cantilever.
D = Downstream edge of cantilever.

TABLE 9

COMPARISON OF PRINCIPAL STRESSES AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		Left abutment				Right abutment			
		Study 3		Study 2-A		Study 3		Study 2-A	
		σP_1	σP_2	σP_1	σP_2	σP_1	σP_2	σP_1	σP_2
3565	U	0	-72	0	+89	0	-44	0	+125
	D	-43	0	+122	0	-40	0	+90	0
3500	U	+18	+45	+9	+121	-1	+149	+7	+126
	D	+204	+51	+235	+62	+306	-22	+336	+85
3450	U	-5	+141	-7	+167	-23	+212	-30	+195
	D	+352	+33	+357	+70	+515	+56	+412	+94
3400	U	+34	+180	+41	+222	-73	+182	-27	+182
	D	+441	+52	+387	+64	+622	+101	+549	+140
3350	U	+45	+198	+61	+236	-21	+253	-27	+229
	D	+482	+67	+523	+36	+522	+38	+615	+22
3300	U	+45	+221	+50	+234	-9	+310	-30	+258
	D	+547	+80	+618	+30	+547	-14	+711	-18
3250	U	+45	+236	+21	+222	-7	+313	+8	+275
	D	+595	+86	+705	+48	+587	-79	+724	+5
3200	U	+58	+249	+5	+237	+43	+313	+24	+294
	D	+611	+83	+731	+68	+631	-73	+719	+14
3150	U	+68	+247	-2	+252	+99	+267	+27	+299
	D	+595	+70	+684	+24	+619	-21	+704	+1
3050	U					+243	0	+256	0
	D					0	+483	-1	+464

Stresses are in pounds per square inch.
(+) Indicates compression, (-) Indicates tension.
U = Upstream face of dam. D = Downstream face of dam.

general decrease in the higher compressive stresses. Tensile stresses were found at the abutments of the top arch; which were probably caused by a larger temperature drop in the concrete than was assumed in the design study. The maximum tensile stress of 79 pounds per square inch and compressive stress of 631 pounds per square inch are well within allowable limits.

ACKNOWLEDGMENTS

It is not possible, in the brief account given here, to mention all those who have contributed to development of Trial Load procedures at the Bureau of Reclamation. This process has benefited from the thought given it by many Engineers. Not the least of these contributions has been made by those who, by the maintenance of the required technical skills, have made possible an effective application of these methods when they were needed.

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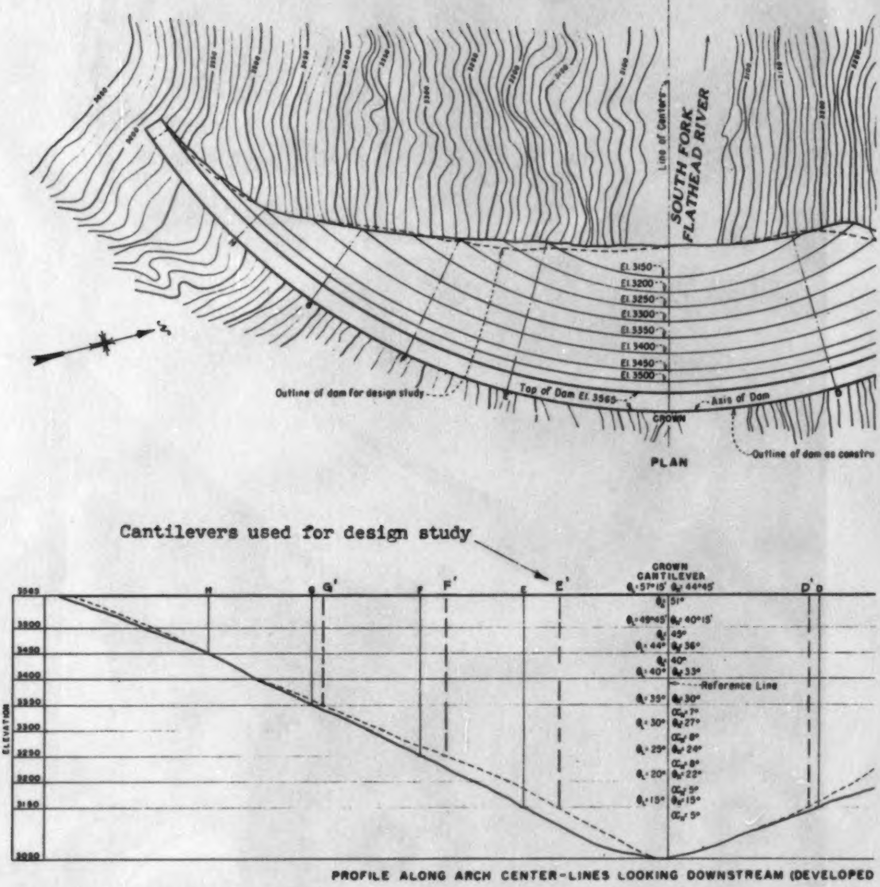
NOTE: References are continued on page 960-25.

FIGURE 1. HUNGRY HORSE DAM


SOUTH FORK
FLATHEAD RIVER
Line of Centers
El 3150
El 3200
El 3250
El 3300
El 3350
El 3400
El 3450
El 3500
Outline of dam for design study
Top of Dam El 3565
Axis of Dam
CROWN
Outline of dam as constru
PLAN
Cantilevers used for design study
CROWN
CANTILEVER
Reference Line
ELEVATION
PROFILE ALONG ARCH CENTER-LINES LOOKING DOWNSTREAM (DEVELOPED

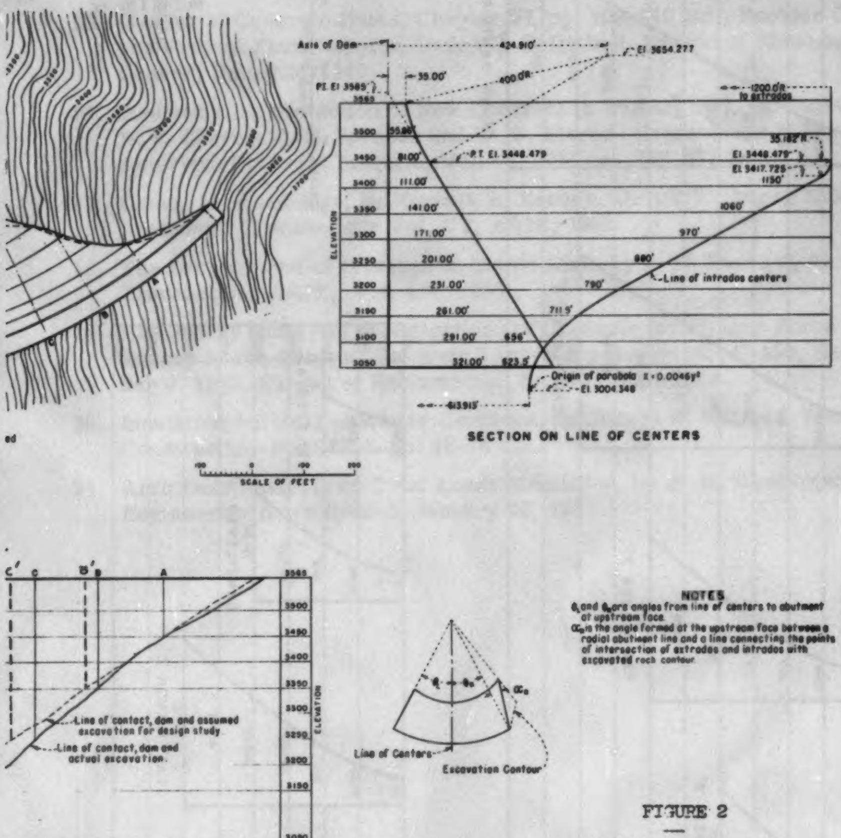


FIGURE 2

HUNGRY HORSE DAM

PLAN PROFILE AND SECTION ON LINE OF CENTERS

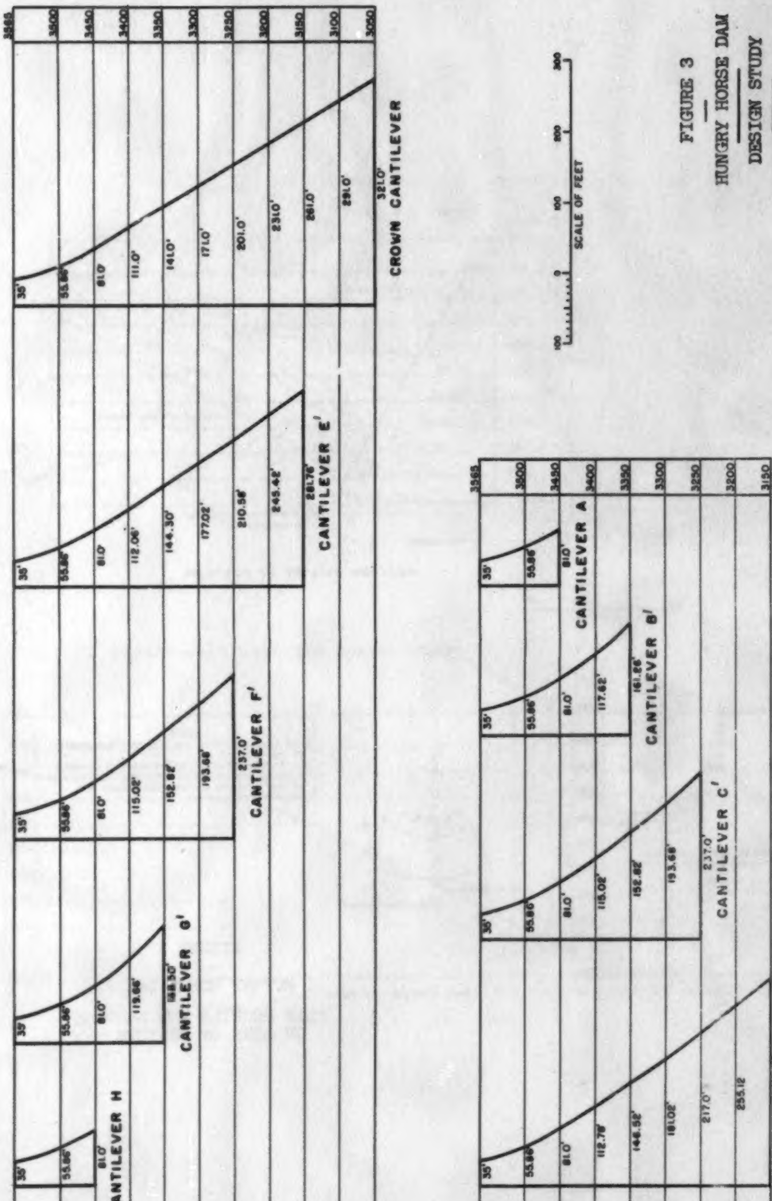


FIGURE 3

HUNGRY HORSE DAM

DESIGN STUDY

CANTILEVER SECTIONS

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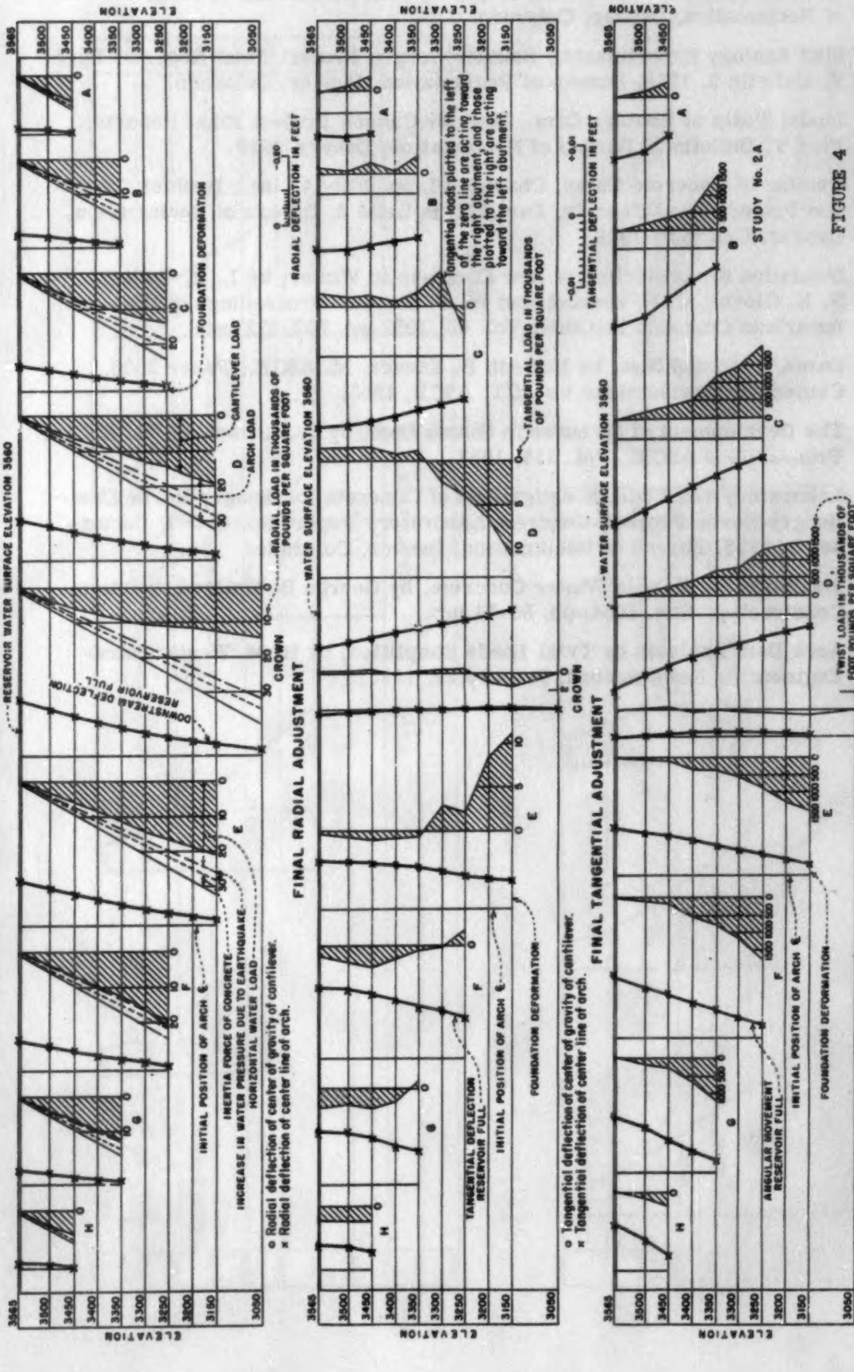

FIGURE 4

HUNGRY HORSE DAM

DESIGN STUDY

RADIAL, TANGENTIAL AND TWIST ADJUSTMENT LOADS

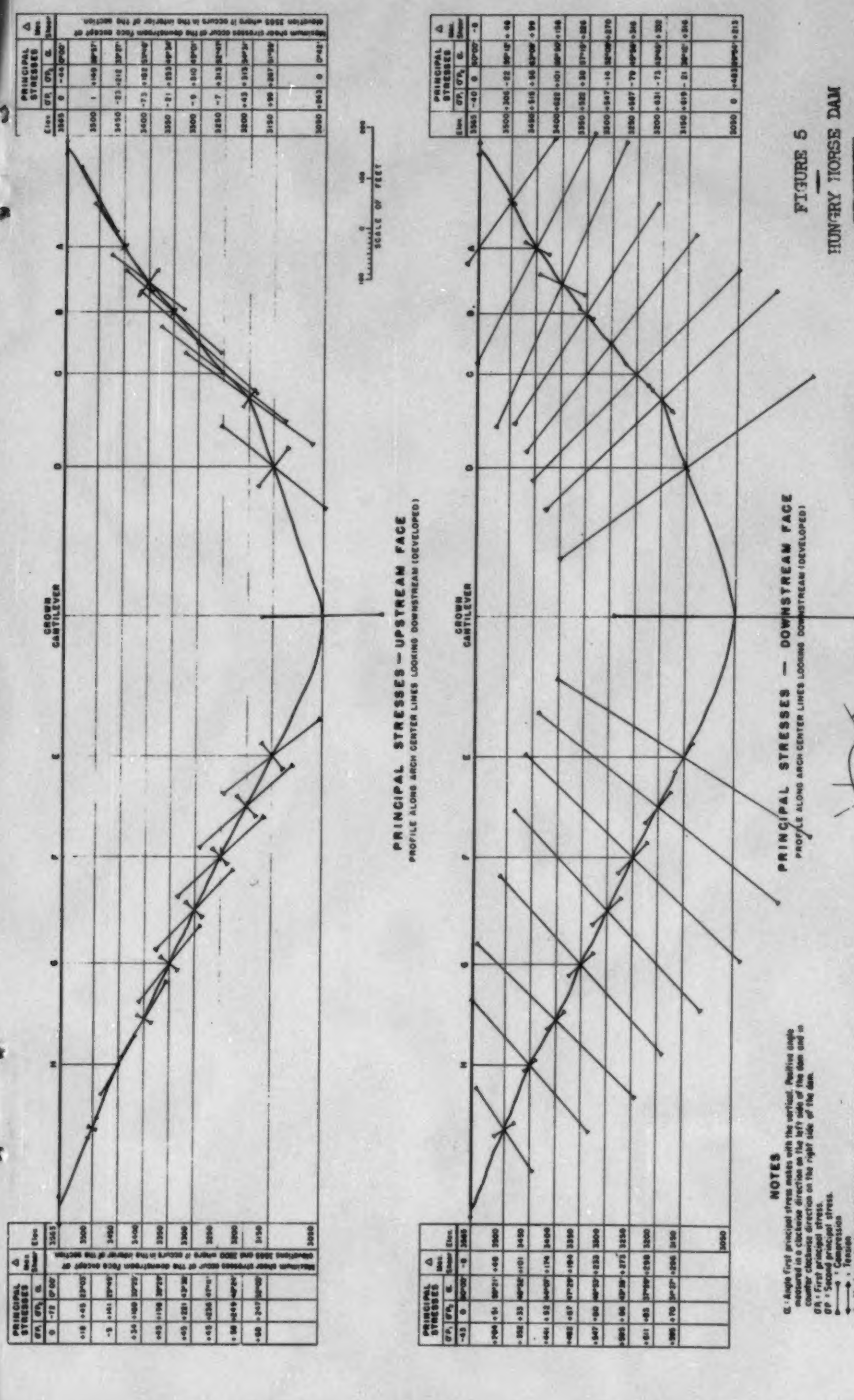

FIGURE 5

HUNGRY HORSE DAM

AS CONSTRUCTED

PRINCIPAL STRESSES INCLUDING EFFECTS OF TANGENTIAL SHEAR AND TWIST

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809 (ST), 810 (HW)[c], 811 (ST), 812 (ST)[c], 813 (ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

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JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1). 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

c. Discussion of several papers, grouped by Divisions.

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